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CONTENTS

Water Requirements and Man-Induced Climate Change
by George H. Hargreaves 247

Furrow Advance Rates under Surge Flow Systems
by A. Alvin Bishop, Wynn R. Walker, Niel L. Allen, and Gregory J. Poole 257

Development of Solutions for Level-Basin Design
by Albert J. Clemmens, Theodor Strelkoff, and Allen R. Dedrick 265

Simplified Method for Rainfall Intensities
by George H. Hargreaves 281

Optimal Design of Border Irrigation Systems
by J. Mohan Reddy and Wayne Clyma 289

Hydraulic Testing of Plastic Filter Fabrics
by Yung Hai Chen, Daryl B. Simons, and Phillip M. Demery 307

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DISCUSSION

Proc. Paper 16480

Optimal Seasonal Irrigation Application Analysis, by William E. Hart, Donald I. Norum, and Gideon Peri (Sept., 1980).
errata 327

Expanded Polystyrene Spheres as Evaporation Suppressors,* by Taeiq A. Mahmoud and Natiq K. Bashi (Sept., 1980).
by Keith R. Cooley 327

Unsteady Drawdown in 2-D Water Table Aquifer,* by David T. Higgins (Sept., 1980).
by George Noutsopoulos and Tilemachos Papathanassiadis 329

INFORMATION RETRIEVAL

The key words, abstract, and reference "cards" for each article in this Journal represent part of the ASCE participation in the EJC information retrieval plan. The retrieval data are placed herein so that each can be cut out, placed on a 3 × 5 card and given an accession number for the user's file. The accession number is then entered on key word cards so that the user can subsequently match key words to choose the articles he wishes. Details of this program were given in an August, 1962 article in CIVIL ENGINEERING, reprints of which are available on request to ASCE headquarters.

*Discussion period closed for this paper. Any other discussion received during this discussion period will be published in subsequent Journals.

16500 INDUCED CLIMATE CHANGE

KEY WORDS: Agricultural economics; **Agricultural engineering; Carbon dioxide; Climatic changes; Environmental effects;** Environmental research; Temperature effects; **Temperature rise; Water requirements;** Water shortage

ABSTRACT: The unrestrained combustion of fossil fuels is possibly the most important environmental issue facing mankind. The concentration of CO_2 in the atmosphere may double within the next 50 years, producing changes in amounts of rainfall, increasing temperatures and changing the agricultural potential. Temperature changes will be greatest (about 8°C) at 83 degrees latitude. A doubling of CO_2 concentrations will increase temperatures near the equator by less than 2°C at latitude 40° by about 3.6°C. In the latitudes 37° to 47° rainfall is projected to decrease while water requirements are estimated to increase by possibly as much as 30 percent. The potential for future agricultural production in the United States will depend largely on the availability of additional water. Quadrupling CO_2 concentrations is projected to increase temperature by more than 6°C at 40° latitude.

REFERENCE: Hargreaves, George H. (Research Engr., Dept. of Agric. and Irrig. Engrg., Utah State Univ., Logan, Utah), "Water Requirements and Man-Induced Climate Change," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR3, **Proc. Paper 16500**, September, 1981, pp. 247-255

16502 FURROW ADVANCE RATES UNDER SURGE FLOW SYSTEMS

KEY WORDS: **Automation; Flow rates; Furrow irrigation;** Furrow systems; **Hydraulic properties;** Intakes; Irrigation design; Time factors; Water flow

ABSTRACT: Two field tests were conducted to study the effect of cycling furrow inflows on advance rates. Earlier investigations had demonstrated the concept and termed it "surge flow" irrigation based on the hydraulic behavior of the cycled, or pulsed, inflow along the furrow. Surge flow treatments were compared with continuous flow tests having the same time-averaged flow rate. All of the tests indicated that surge flow produced significantly faster advance rates. The primary affects were noted during the water application to newly formed furrows with lesser differences in subsequent irrigations. Temporal and spatial variations in furrow intake rates were substantially reduced by the surge flow practices indicating a major advantage of the technique.

REFERENCE: Bishop, A. Alvin (Prof. Emeritus, Dept. of Agr. and Irrigation Engrg., Utah State Univ., Logan, Utah 84322), Walker, Wynn R., Allen, Niel L., and Poole, Gregory J., "Furrow Advance Rates under Surge Flow Systems," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR3, **Proc. Paper 16502**, September, 1981, pp. 257-264

16503 DEVELOPMENT OF SOLUTIONS FOR LEVEL-BASIN DESIGN

KEY WORDS: **Basins; Dimensional analysis; Distribution;** Flow rate; Hydraulics; **Irrigation engineering;** Mathematical models; Open channel flow; Soil moisture; Surface irrigation

ABSTRACT: The procedures are examined for obtaining reasonable estimates of distribution uniformities for a wide variety of flow rates, length of run, infiltration characteristics, and flow resistance for the design and management of level basins. The dimensionless solution of advance and recession in level basins was extended to show the distribution uniformities for a wide variety of conditions. This was then transformed into two representations of distribution uniformity that are more useful for designing and managing level basins. One solution displays the effects of soil moisture deficit and the necessary infiltration opportunity time on distribution uniformity. The other displays the effects of field length and flow rate on distribution uniformity.

REFERENCE: Clemmens, Albert J. (Research Hydr. Engr., United States Water Conservation Lab., 4331 East Broadway Road, Phoenix, Ariz. 85040), Strelkoff, Theodor, and Dedrick, Allen R., "Development of Solutions for Level-Basin Design," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR3, **Proc. Paper 16503**, September, 1981, pp. 265-279

16513 SIMPLIFIED METHOD FOR RAINFALL INTENSITIES

KEY WORDS: Duration curves; Observation; Precipitation (meteorology); Precipitation rate; **Rain and rainfall; Rainfall intensity;** Regression analysis; Rods; **Time factors**

ABSTRACT: Procedures are presented for estimating maximum rainfall amounts for 24 hours with a return period of 10 years from monthly and observation day measurements. Procedures are also presented for converting these values to other rainfall depth- duration amounts. The depth-duration amount varies with the fourth root of the time of duration and the sixth root of the number of years in the return period. Observation-day rainfall is converted to 24-hour values by multiplying by 1.13. Mean monthly rainfall and the maximum monthly values from a 30-year series are used to estimate 10-year return period, 24-hour duration amounts. The 10-year, 24-hour amounts are then used to estimate other depth-duration values. The relationships are good except in protected areas such as for the lee of mountains or in areas surrounded by mountains.

REFERENCE: Hargreaves, George H. (Research Engineer, Dept. of Agri. and Irrigation Engrg. Utah State Univ., Logan, Utah.), "Simplified Method for Rainfall Intensities," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR3, **Proc. Paper 16513**, September, 1981, pp. 281-288

16518 BORDER IRRIGATION SYSTEMS

KEY WORDS: Crop production; Crop yield; **Design; Economic analysis; Irrigation;** Optimization; Simulation models; **Systems engineering;** Water requirements; **Water resources**

ABSTRACT: Using a surface irrigation hydraulics simulation model, relationships were developed between water requirement efficiency and the system design variables. A crop production function was utilized to relate crop yield to the water requirement efficiency. Gross returns from the crop, and the costs of water, labor, ditch construction and crop production were considered in the optimization problem. Several system constraints were incorporated into the design process. The generalized geometric programming technique was applied to the optimal design of border and basin irrigation systems. The design variables were: the length of run, inflow rate into the border, time of inflow, number of lengths of run, width of the border, and the number of border widths in the field.

REFERENCE: Reddy, J. Mohan (Post-Doctoral Fellow, Dept. of Civ. Engrg., Colorado State Univ., Fort Collins, Colo.), and Clyma, Wayne, "Optimal Design of Border Irrigation Systems," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR3, **Proc. Paper 16518**, September, 1981, pp. 289-306

16507 HYDRAULIC TESTING OF PLASTIC FILTER FABRICS

KEY WORDS: Bacteria; **Drainage; Fabrics; Filters;** Ground water; Hydraulics; **Hydraulic tests;** Laboratory tests; **Permeability;** Pipes; Plastics; Sediments; Soils; **Ultrahigh frequencies**

ABSTRACT: The hydraulic characteristics of nonwoven filter filters were investigated using a laboratory permeameter to simulate a soil/fabric subsurface drainage system. By utilizing information of discharge, pressure, permeability, and filter properties, conclusions were drawn concerning the correlation between the soil and the plastic filters. In sands with large concentrations of fine particulate, the migration of the fines was minimal, and the soil controlled the hydraulic response of the system regardless of the filter fabric used. In fine soil where the water velocity is greatly reduced, bacterial activity within the soil can cause erroneous results. Some design criteria are developed for the use of plastic filter fabrics.

REFERENCE: Chen, Yung Hai (Assoc. Prof. of Civ. Engrg., Colorado State Univ., Fort Collins, Colo. 80523), Simons, Daryl B., and Demery, Phillip M., "Hydraulic Testing of Plastic Filter Fabrics," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR3, **Proc. Paper 16507**, September, 1981, pp. 307-324

WATER REQUIREMENTS AND MAN-INDUCED CLIMATE CHANGE

By George H. Hargreaves,[1] F. ASCE

INTRODUCTION

In irrigation project planning, provision is frequently made for dry and wet cycles as well as other extremes. Irrigation facilities, including storage dams, canals, and related structures are often designed for a projected useful life of 50 years. Economic benefits are estimated largely from a review of present yields, prices, and water requirements. Planning and design can be significantly improved if reliable estimates can be made of probable future changes in climatic as well as economic conditions. Until now, little attention has been given to the magnitude or immediacy of possible man-induced climatic changes or the rates at which such changes can be expected to occur.

Although the magnitude of future climate change has not been accurately determined, enough is now known to indicate a need for increasing research efforts and for evaluating new methodologies for monitoring the effects of present and future climatic conditions on food and other agricultural production.

This paper is based on the assumption that increasing use of fossil fuels will produce a fairly rapid global temperature increase due to the increase in CO_2 in the atmosphere. The theory that a cooling cycle is probable is rejected.

From current models of effects that are projected due to increased atmospheric carbon dioxide (models currently being used by the U.S. Dept. of Energy referenced later in this paper) and assuming that the magnitude of predicted temperature increases are approximately correct, an attempt is made in this paper to estimate resulting changes that may occur in water requirements and water management in irrigated and dryland agriculture. Potential evapotranspiration, ETP, is used as an index of water requirements. Justification is presented for using an estimating equation based only on temperature and radiation. Temperature increases predicted by climate models are used together with ETP to estimate an approximate increase in water requirements.

Some proposed research topics relating to water management are presented. The overall purpose of the paper is to promote study, research, discussion,

[1]Research Engr., Dept. of Agr. and Irrigation Engrg., Utah State Univ., Logan, Utah 84322.

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and planning consistent with projected climate changes anticipated during the next 50 years.

Projected Climate Change.—The increasing use of fossil fuels is probably the most important economic and environmental issue facing mankind. During June 19–20, 1980, a workshop was held at the Institute for Energy Analysis, Oak Ridge, Tennessee, on "Climate Change and Food Production in Nonindustrialized Countries." The workshop was sponsored by the American Association for Advancement of Science, AAAS, as part of a project jointly sponsored by AAAS and the U.S. Department of Energy, DOE, on "Environmental and Societal Consequences of a Possible Carbon-Dioxide Induced Climate Change." Some 22 participants from eight countries developed a draft research proposal for submission to DOE by AAAS.

Although many scientists and meteorologists have not yet accepted the conclusion that temperatures will rise significantly during the next 50 years, the workshop did not debate or question this conclusion but rather attempted to develop a proposal for research that would maximize possible benefits to agriculture and food production while also minimizing the undesirable effects.

As indicated by DOE (7), Manabe and Wetherald (6), and Wigley, Jones and Kelly (9) if the use of fossil fuel increases at the current rate, CO_2 in the atmosphere will approximately double within 50 years. Wigley, Jones, and Kelly (9) estimate that doubling CO_2 in the atmosphere will produce an expected increase in global mean temperature of about 2° C. They show that an associated decrease in rainfall will occur for most of North America with the change to warmer years. Manabe and Wetherald (6) developed a simple general circulation model with no seasonal change of insolation and a simplified interaction between cloud and radiative transfer. The computational domain is divided into two equal areas of continent and ocean. It comprises 120° of longitude and 90° of latitude. In their model, predicted precipitation over the continent decreases between latitude 37° and 45° with a doubling of CO_2. Evaporation and evaporative demand increases at nearly all latitudes. Precipitation minus evaporation has the largest negative values between 37° and 47° of latitude. At these latitudes, the mean projected temperature increase for a 100% increase in atmospheric CO_2 varies from 3° C to 3.7° C.

The model, because of its many simplifications and idealizations, should be used with caution. Quantitative details may have significant error on either side of more realistic values. It was developed to provide preliminary insight into the possible climatic response to the increase in CO_2 concentrations and to establish a foundation for a future global model having realistic geography.

The Department of Energy (7) cites a 1975 publication by Manabe and Wetherald (5) and indicates an increase in mean temperature of 2.6° C to 2.8° C at 37° to 47° latitude. They show most of the United States as becoming drier. The following is quoted from the Department of Energy (7):

> It is the sense of the scientific community that carbon dioxide from the unrestrained combustion of fossil fuels is potentially the most important environmental issue facing mankind. Current predictions call for a doubling of atmospheric carbon dioxide as early as 2030. Climate models using these elevated levels, predict the possibility of significant dislocations in the global distribution of climate. Should such perturbations in the distribu-

tion of global temperature, cloudiness, precipitation, and wind occur, then it is clear that there could be major changes in the global distribution of agriculture in the extent and composition of ecological systems, in the biological and physical characteristics of the oceans, and in the extent of the cryosphere. The costs or benefits of such extensive environmental changes to a balanced international societal system are difficult to estimate but could be historically unprecedented. It is therefore of the utmost importance that the technical community resolve the scientific uncertainties concerning the effect of carbon dioxide on climate and the cascade of environmental and societal impacts resulting from a change in climate as quickly as possible.

Temperature Change.—The estimation of future mean temperatures at the earth's surface requires assumptions on the amount of forest clearing, fossil fuel production and use, and the amount of carbon dioxide that will remain in the atmosphere. The possible margin of error in these estimates is considerable. The model prepared by Manabe and Wetherald (6) indicates rather modest changes of 2° C or less near the equator and a maximum increase in temperature at about 83° of latitude. For doubling of CO_2 the maximum increase is 8° C and for quadrupling CO_2 it is shown as 15° C.

The estimated increase is not completely linear but for the range of latitudes of 0° to 65° the temperature increase, T1, in °C, can be estimated reasonably well from latitude, LA, using linear equations. For doubling carbon dioxide the equation is:

$$T1 = 1 + 0.06\,LA \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and for quadrupling carbon dioxide

$$T1 = 3 + 0.08\,LA \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

From these predicted temperature changes it seems probable that a doubling of carbon dioxide in the atmosphere would shift climatic zones for crop adaptation about 250–300 mi (400–480 km) further north. The frost-free growing season would be increased by about one month for latitudes exceeding 30°.

An increase in CO_2 will also increase photosynthesis. Higher temperatures would increase photosynthesis for some crops, particularly the C4 crops or those crops with four carbon atoms on the carbohydrate rings. C4 crops include corn, sugar cane, sorghum, millet, and some of the grasses and forage plants. Water requirements would be increased due to the higher temperatures and the longer growing seasons.

Estimating Potential Evapotranspiration, ETP.—For purposes of this study, potential evapotranspiration, ETP, is taken as an index of water requirements. The question arises as to how ETP will vary with increasing temperature. ETP is also influenced by radiation, and, in latitudes 37° to 47°, rainfall is projected to decrease. The decrease is not great, but, since the amount of radiation reaching the earth depends upon extraterrestrial radiation and upon how much is absorbed and scattered while passing through the atmosphere, radiation may significantly increase. Absorption and scattering of radiation is largely produced by clouds and water vapor. With a decrease in rainfall and cloudiness, an increase in incident solar radiation can be anticipated. However, this increase should not

be large, and, since it would be quite difficult to estimate the magnitude of the increase, it is not taken into account in this study.

The California Department of Water Resources (8) selected Class A pan evaporation from a pan located in a large, irrigated pasture as the most practical and reliable means of estimating ETP. This decision was reached based upon several years of data collection from lysimeters located in irrigated pastures and comparisons with methods of estimating ETP from climatic data. The Technical Committee on Irrigation Requirements of the Irrigation and Drainage Division, ASCE (1), compared 18 methods for estimating ETP using world-wide lysimeter measurements. The Committee report states "Where careful standardization of pan environment is maintained, and where strong, dry-wind conditions occur only occasionally, the use of pans may be as reliable as any of the methods discussed previously." However, the data from California indicate that the use of a large, irrigated pasture eliminates the need for the qualifications stated by the ASCE Committee.

Using five-day sums of grass lysimeter evapotranspiration, ETL, temperature in degrees Fahrenheit, T° F, incident solar radiation, RS, relative humidity, RH, and wind movement, W, from eight years of daily data collected at Davis, California, it was found that RS × T° F predicted 94% of the variance in ETL, $R^2 = 0.94$, for five-day sums. RH in percent predicted 9.2% of the remaining variance and wind only 1.5%, showing that by adding corrections for wind and relative humidity an equation could predict nearly 95% of the variance in ETL. Since other factors did not significantly improve the prediction, an equation was developed using RS and T° F only. The equation can be written:

$$ETP = 0.0075 \times RS \times T° F \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which ETP and RS are in the same units of equivalent water evaporation.

Eq. 3 has been compared with lysimeter evapotranspiration and pan evaporation for eight locations representing a wide variety of climatic conditions in California by using data from the California Department of Water Resources (8). There is little significant difference between accuracy of predicted values from a well standardized Class A pan located in a large irrigated pasture and those from Eq. 3 as a means for estimating ETP.

Although the ASCE Committee has refused to make the lysimeter and other data available for others to use to check methodology, data obtained independently from four locations indicate that Eq. 3 is superior as a means of estimating ETP to any of the 18 procedures tested by the Committee. Hargreaves (2) compares four locations with results from Kohler, Penman, Christiansen, Jensen-Haise and Stephens-Stewart.

Hill (4), in a review of the methods tested by the ASCE Committee, describes all of the methods as being somewhat site specific. If crop coefficients are to be generally applicable, the method for estimating ETP should produce values such that ETP will be zero when ETL is zero. The relationship between ETL and ETP should be approximately linear with a one to one slope. Eq. 3 seems superior to the 18 methods tested if this criteria is used for evaluation.

Most of the equations evaluated had variable intercepts and variable slope relationships when applied to differing lysimeters, indicating that either the equation or crop coefficients require local calibration. Eq. 3 is not site specific.

Mean temperature is used in most evapotranspiration estimating methods.

ETP is principally a daylight phenomenon. At high elevations, the difference between maximum and minimum temperatures is much greater than at low elevations. This introduces a bias in the use of most equations, including Eq. 3, when used for high elevation locations. Also advective energy often accounts for a significantly larger percentage of ETL at higher elevations. Some correction to Eq. 3 may be desirable for higher elevations particularly if the conditions favor the advective transfer of energy.

A major limitation on the evaluation of estimating methods is the paucity of reliable solar radiation measurements. Radiometers vary in type and accuracy. Some are not adequately calibrated or checked for drift. In humid areas, rain and condensation sometimes reduce radiation readings significantly below actual. Preoccupation with including various weather elements in predictive equations will produce limited benefits as long as one of the dominant values of temperature and radiation is frequently measured with an accuracy of plus or minus 10% and sometimes 20% or more.

Hargreaves (3) used the climatic mean values of temperature and estimated radiation from data published by the World Meteorological Organization, WMO, to estimate ETP for 644 world-wide locations. RS was estimated from extraterrestrial radiation, RA, and percentage of possible Sunshine, S. The equation can be written:

$$RS = 0.0075 \times RA \times S^{1/2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

When values of S were not available they were estimated from relative humidity, RH, using the equation:

$$S = KS\,(100 - RH)^{1/2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in which KS is a constant that requires local calibration for each climatic zone and for the hours and methods of measuring relative humidity.

Water Use Changes.—For Mexico, the United States, and Canada, the estimated values of ETP were graphed as a function of latitude, LA. For the latitudes 15° to 65° North, annual ETP in mm for low elevation locations can be approximated by the equation:

$$ETP = 2{,}700 - 40\,LA \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

From Eq. 3 and the estimated temperature increases resulting from the doubling of the concentration of carbon dioxide, an estimate was made of the increase in ETP due to higher temperatures. The results are presented as follows:

Latitude in degrees	0	10	20	30	40	50	60
Increase in ETP, as a percentage	2.5	3	5	7	12	17	26

For latitudes 30° to 60°, there is an added increase in ETP due to the lengthening of the growing season. The projected temperature change would shift present mean temperatures for latitudes 30° to 60° about two degrees further from the equator and increase the length of the frost free season by about 30 days,

resulting in the following percentage increases in the length of the growing season:

Latitude in degrees	20	30	40	50	60
Increase in growing season, as a percentage	0	9	15	25	37

If it is assumed that the percentage increases in ETP for increasing temperature and the percentage increases in length of growing season can be added to indicate a total resulting increase in ETP, then the total increases are as follows:

Latitude in degrees	30	40	50	60
Increase in ETP, as a percentage	16	27	42	63

Problem Area.—For latitudes 37° to 47° in Central United States, the Manabe-Wetherald model indicates a decrease in precipitation and therefore a decrease in soil moisture available for plant growth. Considering both the higher temperature and the increased length of growing season it seems probable that about 30 percent more water would be required during the growing season in order to fully utilize the increased potential for production.

At 42° north latitude, an average annual potential evapotranspiration at low elevations is about 1,000 mm. A 30% increase is equivalent to 300 mm or a depth of 11.8 in. These estimated changes are significant since the major portion of the present corn and livestock production of the United States is located in the potential problem area. Both domestic and world food requirements dictate a need to minimize losses of production from this rich agricultural region. It would seem that immediate attention needs to be given to studies designed to make up this potentially large water deficit through large scale mass water transfer over long distances.

The climate model used and the ETP methods presented herein are not precise and the climatic changes may be significantly greater or considerably less than predicted. At AAAS-DOE workshop, it was generally accepted that Central America would benefit from increased rainfall in the future. However, the amount of rainfall in Central America presents less of a problem than temporal and physical distribution. For much of the Central American Isthmus, the rainy season of six months, May–October, contains a period in July, or August, or both, referred to as the little dry season or the "canicula" or "varanillo." At a regional meeting of hydrologists and meteorologists representing the six countries held in Tegucigalpa, Honduras, in July, 1980 (regional meeting of Comite Regional de Recursos Hidraulicos, Apartado 718 Tegucigalpa, Honduras), an attempt was made to evaluate the effect of a CO_2 induced temperature increase on agricultural production. Based on conclusions of the most complete analysis, an increase in mean temperature would increase annual rainfall but would also increase the amounts during periods that are now excessive and would extend and intensify the little dry season. These changes would increase the need for all kinds of water control, including irrigation, drainage, and flood control.

Potential Benefits

An increase in CO_2 will increase potential plant growth. Higher temperatures will make larger yields possible for some crops. Good lands in northern latitudes will become more productive. If water can be made more available and used more efficiently, the lands receiving less rainfall will have a much greater potential for agricultural production. However, a major problem may be the level of human and animal tolerance to heat.

In the latutides 37°–47°, the projected temperature increase from quadrupling carbon dioxide concentrations is 6°–7° C or an average increase of about 12° F. In order for food production potential to fully benefit from this change, water requirements would be greatly increased over present levels. Plant and animal genetic material would need to be developed or modified to incorporate adjustments to the higher temperatures.

Another possible benefit of potential climatic change studies is that a clearer understanding of the probable increases in temperature will speed up the technological advances required for developing alternatives to the use of fossil fuels. Although potential benefits of climatic change seem large, there is also the question as to how much change can be reasonably tolerated and at what point negative aspects outweigh potential benefits.

Required Research

The AAAS-DOE sponsored workshop selected nine important research topics with various subheadings. These were listed by order of priority. The following topics relating to irrigation and water management are listed as follows with their original priority numbers:

1. The management of water in rainfed agriculture and irrigation frequency.
3. Better understanding of climate/production relationships and collection and use of necessary information.
7. Direct effects of CO_2 on photosynthesis, water use, and nitrogen fixation.
8. Soil temperature and moisture relationship and assessment.

Variability in agricultural production from year to year and from one location to another is influenced more by water than by any other factor or consideration. For this reason, the working group assigned first priority to research related to water management.

Summary and Conclusions

The concentration of carbon dioxide in the atmosphere may double within the next 50 years as a result of increased use of fossil fuels. Temperatures are projected to rise. Photosynthesis and plant growth will generally be increased. Between latitudes 37° to 47°, rainfall is indicated as decreasing. Potential evapotranspiration in these latitudes may increase by as much as 30%. Future agricultural production will become increasingly dependent upon the adequacy of water supplies for irrigation. Climatic zones suitable for specific crops may shift 250–300 mi further from the equator.

With increasing world population and major changes in location and amount of future irrigation land with potential for agricultural production, it will be necessary to revise priorities for water resource development and to reassess the role that the United States can and should have as a major producer and exporter of food.

Toleration of temperatures associated with doubling of carbon dioxide concentrations may not present unmanageable problems for mankind or for plants and animals. However, many adjustments will be necessary. Contemplation of higher temperatures associated with quadrupling of carbon dioxide concentrations raises questions as to the tolerable upper limit for plants, animals and humans. This requires much further research and analysis.

At some levels of change, it appears that modifications induced by increased carbon dioxide concentrations can produce fairly substantial benefits, providing water is made available in deficient areas. Since major man-induced changes are projected within the lifetimes of individuals now living, there is a real need to act quickly to document present climatic conditions more carefully, to reduce the margin of error in estimating the changes and to take the necessary steps to maximize benefits and minimize the possible dislocations and undesirable effects.

In order to model the effects of climate change on world food production, improved information on the water balance is required. Rainfall needs to be estimated during periods that correspond to the capacity of soils to store useful water for crop growth and potential evapotranspiration. Reliable estimates of ETP require accurate measurements or estimates of solar radiation. Therefore, highest priority needs to be assigned to developing world-wide information on present conditions of precipitation, solar radiation, and water balance, all of which provide good spacial coverage. Improved modeling of future water balance conditions can then be undertaken. Without such improved information and models, satisfactory estimates of the probable impacts of climate change on world food production will be impossible.

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Appendix II.—Notation

The following symbols are used in this paper:

ETL = grass lysimeter evapotranspiration;
ETP = potential evapotranspiration, usually in mm;
KS = a constant used in estimating sunshine percentage from relative humidity;
LA = latitude in degrees (in this report North latitude);
RA = extraterrestrial radiation in equivalent mm of water evaporation;
RH = relative humidity in percent;
RS = incident solar radiation in equivalent mm of water evaporation;
S = percentage of possible sunshine;
T° F = mean temperature in degrees Fahrenheit;
T1 = temperature increase (resulting from an increase of CO_2 concentrations in the atmosphere); and
W = wind movement in distance per unit time.

Furrow Advance Rates under Surge Flow Systems

By A. Alvin Bishop,[1] Wynn R. Walker,[2] Members, ASCE, Niel L. Allen,[3] and Gregory J. Poole[4]

Introduction

Considerable attention has been given to the advance phase of surface irrigation because of its bearing on intake opportunity times between the two ends of the irrigation run. Surface irrigation design and management objectives are generally to complete the advance phase of the irrigation as quickly as possible to minimize these differences in intake opportunity time. To accomplish this, large streams have been recommended for the advance phase with cutback streams or runoff recovery systems to minimize the runoff and deep percolation losses during the intake phase. The ideal system would be capable of automation, provide instantaneous advance, and prevent runoff during the intake phase. Unfortunately, most surface irrigation systems, particularly sloped furrow systems, do not have the physical resources needed for achieving these conditions without substantial capital investment.

Surge flow for automatic furrow irrigation was first suggested by Stringham and Keller (6) as an improved method of automating cutback furrow irrigation. Since most simple automatic irrigation valves can only turn water on and off, they concluded that it would be simpler to cycle valves to reduce the flow rate on a time basis than to partially close the valves to achieve the cutback stream. A cycling flow system complete with valves and an automatic controller was developed and successfully tested to accomplish pulsed application of water to the furrows. The field testing of the flow equipment indicated that cutback could be achieved to almost any desired average stream size using time cycles from a few seconds to hours in duration. Perhaps more importantly, it was found that the cycled flows had a major impact on furrow advance rates as well as the temporal and spatial intake variability within the field.

[1]Prof. Emeritus, Dept. of Agr. and Irrigation Engrg., Utah State Univ., Logan, Utah 84322.

[2]Assoc. Prof., Dept. of Agr. and Irrigation Engrg., Utah State Univ., Logan, Utah 84322.

[3]Irrigation Engr., Pitcher Irrigation Co., Preston, Idaho.

[4]Hydr. and Irrigation Engr., Wright-McLaughlin Engrg., Denver, Colo.

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Definition of Surge Flow

The intermittent application of irrigation to surface irrigation systems is distinctly different than with either sprinkle or trickle systems. Karmeli and Peri (4) described the pulsing of sprinklers to achieve reduced time averaged application rates. In a furrow system, however, a pulse of water introduced at the furrow inlet is subject to the various resistance and inertial forces as it moves along the furrow, and therefore elongates and slows down. An important result of these dynamic factors is that the advancing tip moves more rapidly than the receding edge. Thus, when the intervals between water inputs shorten, the pulses eventually merge together some distance along the furrow. In fact, during the early testing by Stringham and Keller (6), the flow at the end of the furrows was relatively steady. The behavior of pulsed furrow inflows is better characterized by surge hydraulics, as defined by Chow (3). The term surge flow was therefore adopted and is still conceptually correct when time intervals between surges is great enough so that individual impulses do not interact.

Surge flow creates a series of on and off conditions of constant or variable time spans at the furrow inlet. The cycle time has been defined as the period required for a complete on/off cycle, i.e., the time between the beginning of one surge to the beginning of the next. The cycle time may be of any desired duration and can vary from a few seconds to hours, but present experience indicates a typical cycle time of 10 min–60 min. The cycle ratio is defined as the ratio of the on time to cycle time, e.g., with conventional irrigation, a furrow set of 6 h might be considered to have a 6-h cycle time with no off time, making the cycle ratio equal to one (continuous flow). This same irrigation might be accomplished with the same furrow stream and an irrigation time of 12 h using a 30-min cycle time and a cycle ratio of one-third, i.e., 10 min on and 20 min off in each cycle. The applied water would be reduced by one third, reducing the on time for each furrow from 6 h to 4 h, but allowing a more convenient "set," or total irrigation time. The number of furrows per set would be increased or decreased proportionately.

Experimental Procedures

Field studies were carried out during the 1979 irrigation season to test the characteristics of surge flow irrigation in furrows with particular emphasis on the advance phase. The third writer (1) used instantaneous furrow streams of 10 gpm (0.63 L/s), 20 gpm (1.26 L/s), and 30 gpm (1.89 L/s) with cycle ratios of one for the 10 gpm (0.63 L/s) inflow, one half for the 20 gpm (1.26 L/s) stream and one third for the 30 gpm (1.89 L/s) stream, resulting in an equal quantity of water being applied to each furrow over a given time period. The cycle time for these tests was 10 min. The fourth writer (5) used surge flow furrow streams of 20 gpm (1.26 L/s) with cycle times of 2 min, 5 min, 10 min, and 20 min. In each case the cycle ratio was one half, making the time averaged flow stream 10 gpm (0.63 L/s) and equal to a continuous flow of 10 gpm (0.63 L/s) to furrows irrigated at the same time for comparison.

The studies were carried out on the Utah State University Dairy Research Farm located about 1 mile north of the Utah State University campus. The

soil is classified as a silt loam and the site was planted to corn with a 30 in. (0.76 m) row spacing. The furrows were 600 ft (183 m) long and had an average slope in the direction of irrigation of 1.46%. Planting and cultivation were planned so that tractor wheels always ran along the same furrows, resulting in a compactive effort to every other furrow. The compacted furrows were designated "wheel furrows" and those not compacted by the tractor wheels were designated as "non-wheel furrows." The tools used to form the furrows before the first irrigation covered the wheel tracks and left all furrows with similar roughness and cross section. Prior to making a test run, 100 ft (30.5 m) stations were established along the furrows to aid in measuring the advance rate. In addition, soil moisture samples were taken from depths of 0.5 ft, 1.0 ft, 2.0 ft, and 3.0 ft (150 mm, 300 mm, 600 mm, and 900 mm) for every other test furrow.

Using the equipment described by Stringham and Keller (6), water was applied to randomly selected furrows in the cycle times and cycle ratios outlined previously. During each test, the time for water to advance to each station along the furrow was recorded. Furrow runoff was also measured so that intake could be determined from the inflow-outflow data. Since very little information exists regarding surge flow, several exploratory runs were made before choosing the cycle times and cycle ratios decided upon.

Several replications of each treatment were conducted over the field and it was initially assumed that spatial variability could be neglected. Consequently, all of the advance data were first fitted to a power advance function and then "averaged" by treatment to produce a single relation to each cycle time and cycle ratio combination. The mean relation was determined by numerically averaging the logrithmically transformed power advance coefficients.

Results

The effects of surge flow irrigation were most apparent during the first irrigation when the hydraulic and infiltration characteristics were extreme. Fig. 1 shows the advance data for the first irrigation in non-wheel furrows on July 3, 1979 (1). The average flow rate is 10 gpm (0.63 L/s) for a continuous flow treatment and three surge flow treatments utilizing a 10 min cycle time and cycle ratios as shown. It can be seen that the continuous flow treatment required almost an order of magnitude more time to complete the advance than the surge flow irrigated furrows. Although the difference between the continuous and surge flow treatments is statistically significant at the 5% confidence level, the differences among the surge flow treatments were not. Fig. 2 shows the results of the same tests for the second irrigation. The advantages of surge flow were substantially reduced. The differences in surge flow advance rates are not greatly different but the continuous flow produced much faster advance.

In each case previously noted for the third writer (1) data, the time average furrow inflows were equal to 10 gpm (0.63 L/s). To eliminate the effects of variable instantaneous flow rates, the fourth writer (5) introduced 10 gpm (0.63 L/s) into the continuous flow furrows and into each surge flow furrow but varied the cycle time. The cycle ratio was fixed at one half. Again, the most pronounced effects of surging the furrow inflows occurred during the first irrigation in the wheel furrows (Fig. 3). It can be seen that the advance under

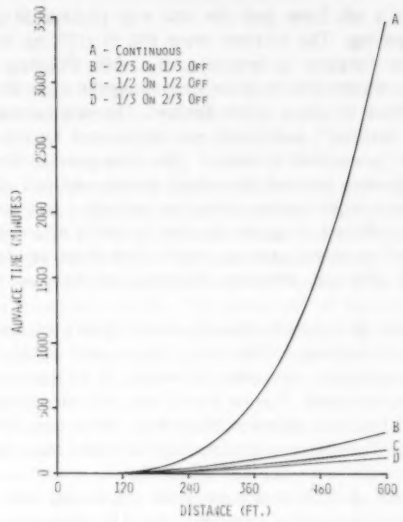

FIG. 1.—Advance Rates under Continuous and Variable Cycle Ratio Surge Flow Treatments of Newly Formed Non-Wheel Furrows (1 ft = 0.305 m)

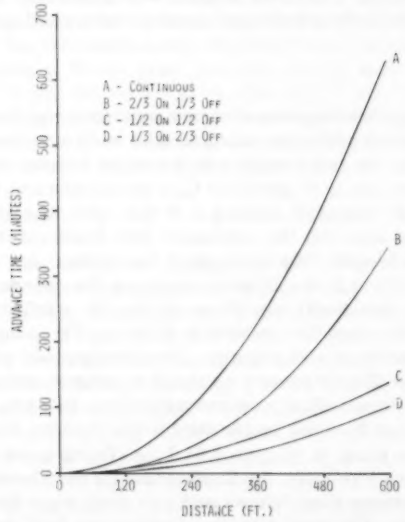

FIG. 2.—Advance Rates for Continuous and Variable Cycle Ratio Surge Flow Treatments of Non-Wheel Furrows During Second Irrigation (1 ft = 0.305 m)

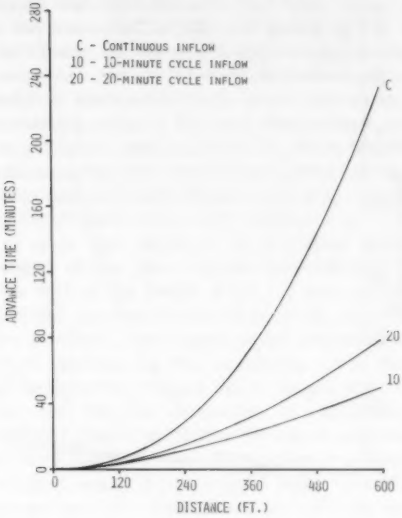

FIG. 3.—Advance Rates in Newly Formed Wheel Furrows under Continuous and Two Variable Cycle Time Surge Flow Treatments (1 ft = 0.305 m)

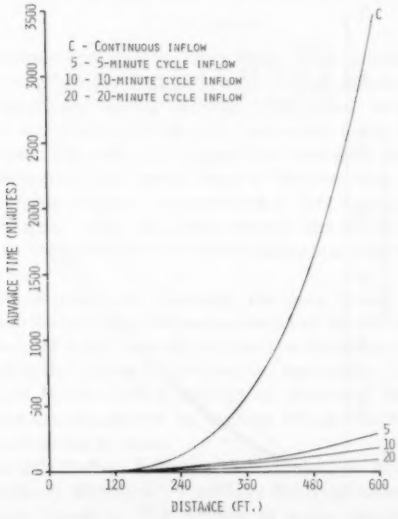

FIG. 4.—Advance Rates in Newly Formed Non-Wheel Furrows under Continuous and Variable Cycle Time Surge Flow Treatments (1 ft = 0.305 m)

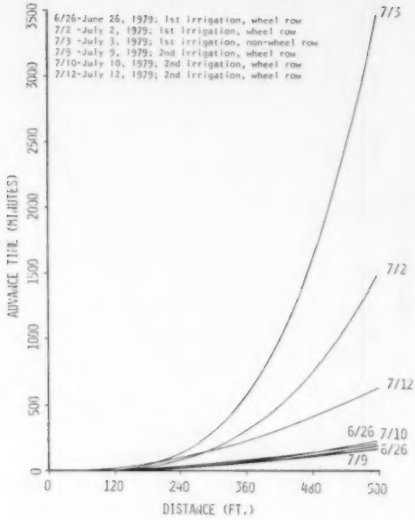

FIG. 5.—Advance Relations for Various Continuous Flow Furrow Irrigations in 1979 (1 ft = 0.305 m, 10 gpm = 0.67 L/s)

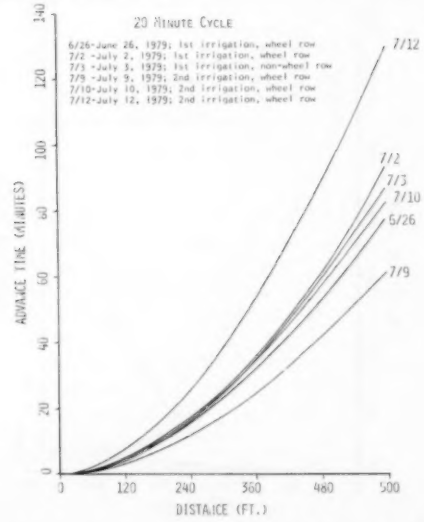

FIG. 6.—Advance Relations for Surge Flow Furrow Irrigations in 1979 (1 ft = 0.305 m, 10 gpm = 0.63 L/s)

surge flow conditions was approximately four times faster. Results from the first irrigation in the non-wheel furrows are shown in Fig. 4. After the first irrigation when infiltration differences should be less noticeable in the field or when wheel compaction reduces these differences mechanically, the advance under surged conditions were much closer to the rates under continuous flow. It appears that introducing a furrow flow and then letting it recede and infiltrate before the next surge passes significantly affects the permeability of the furrow perimeter. Given the tendency for newly formed furrows to have widely different intake characteristics than previously irrigated furrows, it also appears that surge flow systems may significantly reduce these differences.

These effects of surge flow practices on infiltration rates are probably the most important aspect of this new irrigation methodology. To illustrate these effects further, the data of the fourth writer (5) were grouped by continuous treatment in Fig. 5 and for the 20 min cycle in Fig. 6. While the shortening of the advance time is evident, as previously noted, a more interesting observation is the total variation between the two treatments. Over the season, over the field, and among replications, advance times ranged from 270 min–3,490 min for the continuous flow furrows, irrespective of compactive history. For the 20 min cycle, the range varied from 60 min–130 min. In other words, the variation under continuous flow conditions was approximately a factor of 13, whereas for the 20 min cycle it was only a factor of about two. If further testing of the surge flow concept verifies a significant reduction in the temporal and spatial variability of intake, surge flow systems will be amenable to much more precise design and operation strategies than for conventional furrow irrigation.

Conclusions

The research reported herein concerns a single field condition and a single time average furrow inflow rate. Instantaneous furrow inflows were varied by conducting surge flow tests having variable cycle ratios. In all trials to date, the advance phase was accomplished with less water using surge flows than with continuous flows. The effects of surge flow were most pronounced during the first water application to newly formed furrows and were also more pronounced in non-wheel furrows. A comparison of data aggregated by treatment but not time or location within the field showed that differences in advance rates under surge flow conditions were substantially less than under continuous flow conditions.

In conducting these tests and evaluating the data, it has become obvious that surge flow alters the basic intake characteristics of the furrow. It is theorized that cycled movement of water over the soil surface accelerates the development of thin surface seals in the bottom of the furrows. Apparently, during the interval between surges, the bottom soil consisting of dispersed fine materials and lubricated by water are compacted by tension forces which build up in the soil as the furrow continues to drain.

The limited research to date indicates that surge flow has the potential of a powerful management technique to improve the application uniformity and efficiency of furrow irrigation. The volume of water necessary to complete the advance phase can be substantially reduced using surge flow over the conventional practices. Likewise, the intake opportunity time between the head

end and the far end of the run can be equalized, resulting in a more uniform water distribution. Finally, the possibility of programming changes in the cycle ratio to achieve cutback can effectively eliminate runoff with its water loss and irrigation return flow problems.

It is evident, however, that much additional study is required to establish the design criteria for the wide range of soils, length of run, size of stream, and other conditions of surface irrigation. The effect of surge flow on intake, advance, uniformity, and efficiency must be determined. Theoretical work on the dynamic geometry of the surge and its relationship to intake and advance is necessary.

Acknowledgment

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DEVELOPMENT OF SOLUTIONS FOR LEVEL-BASIN DESIGN

By Albert J. Clemmens,[1] A. M. ASCE, Theodor Strelkoff,[2] M. ASCE and Allen R. Dedrick[3]

INTRODUCTION

Level-basin irrigation has grown rapidly in popularity over the last decade. There are several reasons for its popularity: (1) Uniform application of water and associated high application efficiencies, which reduce farm water requirements and decrease nutrient and fertilizer loss from excessive deep percolation; (2) elimination of runoff, which reduces soil erosion and eliminates water reuse systems; and (3) better control of the irrigation water, thus reducing labor requirements. The development of laser-controlled land leveling equipment has greatly improved the achievable efficiencies for level basins.

In level-basin irrigation, water enters the field at one end or at a corner and spreads over the basin. The driving force is the water surface slope—the higher the flow rate, the greater the flow depth and water surface slope, and the faster the rate of advance. Once the entire basin is inundated the water surface levels out. The irrigation is over when all of the water infiltrates into the ground (recession). This recession occurs at about the same time over the entire basin. The difference in opportunity time from one part of the field to the other is essentially the difference in advance time. Since the rate of advance increases as flow rate increases, the uniformity of application will also increase with flow rate.

Level-basin irrigation is well suited to low intake soils since the difference in infiltrated depth for a given difference in opportunity time is relatively small. Also, advance across the basin is faster for low intake soils than for higher intake soils since more water remains on the surface, increasing the water surface slope. This reduces the problem of balancing stream size and application time commonly associated with sloping-border irrigation. This paper addresses proce-

[1]Research Hydr. Engr., United States Water Conservation Lab., 4331 East Broadway Road, Phoenix, Ariz. 85040.

[2]Formerly Prof. of Water Sci. and Civ. Engrg., Land, Air and Water Resources, Water Sci. and Engrg. Section, Univ. of California, Davis, Calif.

[3]Agricultural Engr., United States Water Conservation Lab., 4331 East Broadway Road, Phoenix, Ariz. 85040.

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dures for obtaining reasonable estimates of distribution uniformities for a wide variety of flow rates, lengths of run, infiltration characteristics, and flow resistance for the design and management of level basins.

Level Basin Design

In designing level basins, there are many options to consider depending on imposed limitations of stream size, basin size, soil infiltration characteristics, and crop management considerations. In many cases, there is a maximum practical stream size depending on the delivery source and field turnouts. Erosion becomes a problem with high flow rates so the water must be spread out over a large area using several small turnouts, erosion control structures, or both. Maximum water depth can also impose a limitation on stream size, particularly with narrow basins or row crops. Basin size is often limited by physical constraints, such as roads, property lines, canals, etc.

The Soil Conservation Service (SCS) has developed a design manual for border irrigation (20). A series of design charts are presented for level-basins and sloping borders. The charts presented for level basins relate unit flow rate and length of run for various application efficiencies. A range of Manning roughnesses, soil infiltration families, and net depth of application are presented in the charts. Kruse defined the term application efficiency (E_a) as the ratio of the average depth of the irrigation water stored in the root zone to the average depth of irrigation water applied (13). If the irrigation completely fills the root zone, the application efficiency for a unit width border is

$$E_a = \frac{z_d L}{q_{in} t_{co}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which L = the field length; q_{in} = the flow rate per unit width; t_{co} = the application time or time of cutoff; and z_d = the soil moisture deficit or the depth required to fill the root zone. It is also useful to define the distribution uniformity for blocked end borders or basins as

$$DU = \frac{z_n L}{q_{in} t_{co}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which z_n = the minimum or net depth infiltrated. The relationships between z_n, z_d, DU, and E_a are shown in Fig. 1 where z_g is the gross or average depth applied, $z_g = q_{in} t_{co}/L$. When the minimum depth infiltrated is less than the desired depth ($z_n > z_d$), then $DU > E_a$ [Fig. 1(*a*)], and the entire field has been over irrigated. When $z_n = z_d$, then $DU = E_a$, and E_a is a maximum for an adequate irrigation [Fig. 1(*b*)]. However, when $z_n < z_d$, then $E_a > DU$ and part of the field has been underirrigated. Note that for this case, E_a is not easily calculated [Fig. 1(*c*)]. A similar analysis can be made for irrigation efficiency, which includes other beneficial uses of water.

The SCS solution assumes that $z_n = z_d$ or $DU = E_a$ and the soil moisture deficit is just met at the end of the field. The solutions developed in this paper use DU, since it is more straightforward for all three cases. Conceptually, the

case in Fig. 1(*b*) is the most desirable, and for design purposes we can determine DU and use it as the potential E_a. In general, the distribution uniformity is dependent on the irrigation system design. Application efficiency depends on both design and irrigation management and is an individual irrigation and a seasonal parameter. The distribution uniformity can be thought of as the upper limit of application efficiency (when not considering deficit irrigation or net soil moisture depletion over the growing season).

In well designed systems, both sloping (without pumpback) and level, higher efficiencies are usually attained if the incoming stream is cut off before the entire field has been covered with water. However, the SCS solution method for level basin design assumes that advance is complete before cutoff. They also make several other arbitrary assumptions regarding the friction slope and water depths (5). Also, the SCS intake families, in principle, couple the magnitude

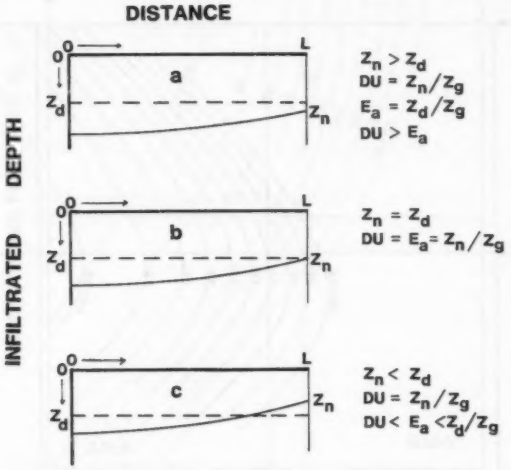

FIG. 1.—Profiles of Infiltrated Water: (*a*) For Overirrigation Throughout Basin; (*b*) For Adequate Irrigation with No Excess Overirrigation; and (*c*) For Slight Underirrigation over Part of Field

and shape of the infiltration function which do not always represent the true infiltration of the soil.

Merriam (14), as well as others, emphasized that the time required to infiltrate the desired depth of application of soil moisture deficit is a key factor in designing border-irrigation systems. For analyzing distribution uniformity, it is more useful to use the minimum opportunity time, τ_n, which corresponds to z_n.

Once the basins have been designed and built, management of the system consists of determining the application time necessary to apply a desired volume of water (or gross depth of application). The application time or time of cutoff can be found from Eq. 2, provided that the flow rate is known and an accurate estimate of the distribution uniformity is obtainable.

The appropriate factors which govern the flow in irrigation borders or basins are the inflow rate per unit width (q_{in}), the flow resistance (Manning n), time of cutoff or application time (t_{co}), field length (L), field slope (S_o), the field end conditions (blocked end or open ended), and the infiltration characteristics of the soil.

Field evaluations of all the possible combinations of the aforementioned factors would be virtually impossible. For this reason, it is desirable to use a mathematical model of the system to determine the results of different combinations of these variables. Many models have been proposed and developed to describe the flow of water in irrigation borders. Some are based on empirical approaches,

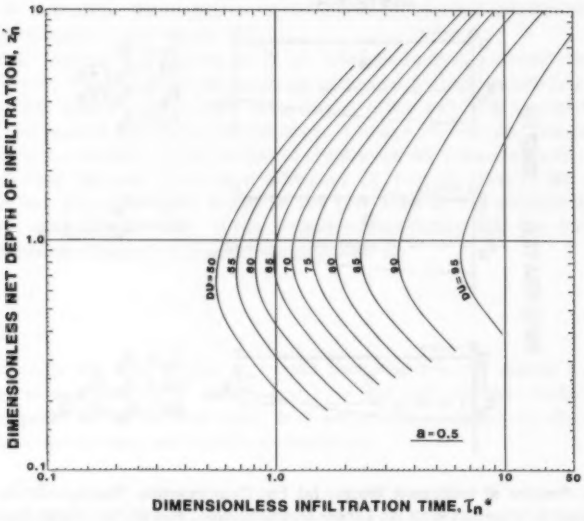

FIG. 2.—Level-Basin Distribution Uniformities Representing Variations in Infiltration and Soil Moisture Deficit, $a = 0.5$

volume balance techniques, or simplified hydraulic models (2,8,14,15,16,17,21,22). These methods depend on normal depth in the surface stream, thus they do not apply to the level basin case. Other models are based on exact mathematical equations for flow in open channels, i.e., the Saint Venant equations (1,10,12). While highly accurate, these models are time consuming and expensive to run. Recently, a new "zero-inertia" model, based on the Saint Venant equations has been developed (19). This "zero-inertia" model is accurate and relatively reliable, but inexpensive to run. Verification of the model was presented by Strelkoff and Katopodes (10) and Clemmens (3,4).

Given the unit inflow rate and cut-off time, and the length, roughness, and infiltration characteristics of the basin, the mathematical model is capable of predicting the advance and recession of the surface stream, and thus, the post-irrigation profile of infiltrated water over the length of the basin. These advance and recession data were used to develop the uniformity curves presented in this paper. Given the minimum depth of application z_n, the distribution uniformity, DU, can be computed (as can other measures of the merit of a given choice of input variables): $DU = f(z_n, q_{in}, t_{co}, L, n, k, a)$ in which k and a are the constant and exponent of a power infiltration function which will be defined in a later section. The variable z_n is not an independent variable; indeed $z_n = \phi(q_{in}, t_{co}, L, n, k, a)$. Therefore $DU = \psi(q_{in}, t_{co}, L, n, k, a)$. Many

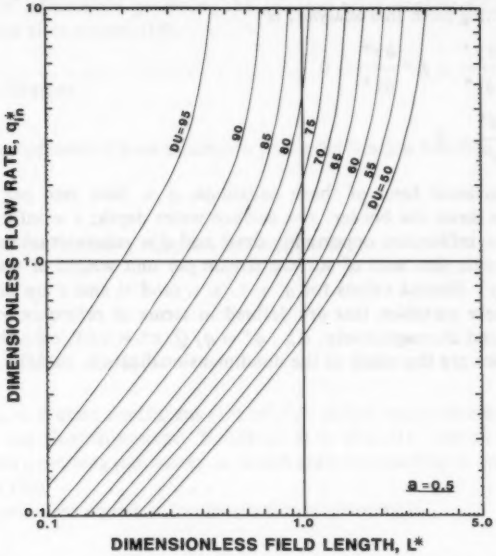

FIG. 3.—Level-Basin Distribution Uniformities Representing Variations in Flow Rate and Field Length, $a = 0.5$

computer runs, with systematic variation of each of the independent variables (input parameters) would show the effect on distribution uniformity. However, an astronomical number of runs and graphs of results would be required. The number of runs and output graphs is reduced to a manageable size by using dimensionless notation. It will be shown in the following sections, that in terms of dimensionless variables $DU = f_1(z_n^*, \tau_n^*, a)$ in which $z_n^* =$ the minimum depth of infiltration relative to a characteristic depth; and $\tau_n^* =$ the time required to infiltrate a depth z_n, relative to a characteristic time. This function, plotted in Fig. 2 for $a = 0.5$, can be used to show how the minimum infiltrated depth and infiltration characteristics of the soil (as well as field length, roughness, and inflow rate) affect the distribution uniformity.

Alternately, it will be shown that $DU = f_2(q^*_{in}, L^*, a)$ which shows, with a different set of dimensionless terms, the effect of inflow rate and field length (as well as minimum depth, field roughness, and infiltration characteristics) on the distribution uniformity. This function is plotted, for $a = 0.5$, in Fig. 3. Illustrative examples show how these curves can be used to determine suitable values of inflow rate, cut-off time, and basin dimensions.

Basic Dimensional Analysis

The second and first writers (18) presented a detailed dimensional analysis of the variables associated with both sloping-border and level-basin irrigation. The dimensionless, differential form of the Saint Venant Equations for level basins, resulting from this analysis, is

$$\frac{\partial q^*}{\partial x^*} + \frac{\partial y^*}{\partial t^*} + K^* \frac{\partial z^*}{\partial \tau^*} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$\frac{\partial y^*}{\partial x^*} - D^* \frac{d^*}{y^*} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

In the dimensional form of these equations, q = flow rate per unit width; x = distance down the border; y = surface water depth; z = infiltrated depth; t = time; τ = infiltration opportunity time; and d = relative resistance to flow, or drag per unit plan area of surface stream per unit weight of water (friction slope times y). Starred values for q, x, y, z, t, (and τ) and d are dimensionless values of these variables that are defined in terms of reference variables, Q, X, Y, Z, T, and D, respectively, e.g., $q^* = q/Q$, $x^* = x/X$, etc. The remaining three variables are the result of the nondimensionalization, namely

$$K^* = \frac{QT}{XY} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

$$K^* = \frac{Z}{Y} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

$$D^* = \frac{\dfrac{D}{Y}}{\dfrac{Y}{X}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Boundary conditions for the solution of Eqs. 1 and 2 for basin irrigation are

$$q^* = q^*_{in}; \quad 0 < t^* \le t^*_{co}; \quad \text{at} \quad x^* = 0$$

$$q^* = 0; \quad t^* > t^*_{co}; \quad \text{at} \quad x^* = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

$$q^* = 0 = y^*; \quad x^* = x^*_A < L^* \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

$$q^* = 0 = y^*; \quad t^* = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

$$q^* = 0; \quad x^* = L^* \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

$$0 \le x^* \le L^* \quad \ldots \quad (12)$$

in which q_{in} = the unit flow rate at the head of the basin; t_{co} = the time of cutoff or application time; x_A = the advance distance; and L = the field length.

The solution of Eqs. 1 and 2 depends upon values chosen for $\forall^*$, K^*, D^*, q_{in}^*, t_{co}^*, and L^*. The solutions for level-basin advance presented by the first and second writers (6) set $\forall^* = 1$, $D^* = 1$, $q_{in}^* = 1$, and $t_{co}^* = 1$, without loss of generality. Furthermore, it was postulated that $L^* = \infty$. Finally, the characteristic infiltration depth was defined as $Z = z(T)$. The solution was governed by K^* and an additional parameter resulting from selection of the infiltration function. The number of parameters that can be arbitrarily set and the number of parameters governing the solution were considered in detail by the second and first writers (18).

Dimensionless Systems

The solution presented here assumes a power infiltration function of the form

$$z(\tau) = k\tau^a \quad \ldots \quad (13)$$

and a form of the Manning equation for roughness

$$d(y,q) = \frac{q^2}{\left(\frac{C_u}{n}\right)^2 y^{7/3}} \quad \ldots \quad (14)$$

in which C_u = a units coefficient (1.0 $m^{1/2}/s$ in the metric system and 1.486 $ft^{1/2}/sec$ in the English system). Similarly, $D = d(Y,Q)$. Use of Eq. 13 adds one additional governing parameter, a, which must be specified in the dimensionless solution (18).

The dimensionless equation for distribution uniformity is

$$DU = \frac{z_n^* L^*}{q_{in}^* t_{co}^*} K^* \forall^* \quad \ldots \quad (15)$$

with D^* and a the only other possible governing parameters governing DU. The solution of Eqs. 3, 4, and 8–12, with $\forall^*$, K^*, a, D^*, q_{in}^*, t_{co}^*, and L^* all fixed, leads to a distribution of infiltrated depths resulting from the advance and recession of the irrigation stream. Thus, for each set of input conditions, a value for z_n^* can be obtained and DU calculated from Eq. 15. The resulting solution for DU would be governed by a and two other dimensionless parameters. For the dimensionless system, used by the first and second writers (6), in which $D^* = \forall^* = q_{in}^* = t_{co}^* = 1$, the solution for DU would be governed by input values for K^*, a, and L^*. Alternately, the solution for DU could be presented as a function of z_n^*, a, and L. At first glance, it appears that this direct solution is useful. The values of X, Y, and Z would be given by Eqs. 5–7 with Q

$= q_{in}$, $T = t_{co}$ and $V^* = D^* = 1$ as follows

$$X = \frac{q_{in}^{7/16}\, t_{co}^{13/16}}{\left(\frac{n}{C_u}\right)^{3/8}} \quad \ldots\ldots (16)$$

$$Y = q_{in}^{9/16}\, t_{co}^{3/16} \left(\frac{n}{C_u}\right)^{3/8} \quad \ldots\ldots (17)$$

$$Z = k t_{co}^{a} \quad \ldots\ldots (18)$$

From Eq. 15, with $q_{in}^* = 1$, $t_{co}^* = 1$, $V^* = 1$,

$$DU = z_n^*\, L^*\, K^* \quad \ldots\ldots (19)$$

Obtaining the dimensional equivalent of this form of dimensionless solution would require that q_{in}, t_{co}, k, a, and n all be known a priori. This is impractical for design and management purposes, since t_{co} is not known in advance. It would be more practical if t_{co} were a dependent, rather than an independent, variable. Thus, for a useful solution, t_{co}^* should not be one of the fixed parameters.

The possible choices for an alternate fixed parameter to replace $t_{co}^* = 1$ are $K^* = 1$, $z_n^* = 1$, and $L^* = 1$. Selecting $z_n^* = 1$ is not practical since z_n^* is a result of the solution for advance and recession and is not known a priori. Thus, a trial and error solution would be necessary to obtain $z_n^* = 1$. Selecting $L^* = 1$ was also considered by the first and second writers (6). They did not select it, since advance after cutoff could be displaced more easily by setting $t_{co}^* = 1$. However, in this manuscript, where $t_{co}^* = 1$ is not desirable, selection of $L^* = 1$ is a logical choice. To avoid confusion, the following subscript notation was adapted to distinguish between dimensionless systems: in the work of the first and second writers (6), $Q_A \equiv q_{in}$ and $T_A \equiv t_{co}$; for a solution $L^* = 1$, $Q_B \equiv q_{in}$ and $X_B \equiv L$. Solving for the reference variables from Eqs. 5–7 gives (with $V^* = 1$, $D^* = 1$, $Z = kT^a$)

$$Y_B = q_{in}^{6/13}\, L^{3/13} \left(\frac{n}{Cu}\right)^{6/13} \quad \ldots\ldots (20)$$

$$T_B = \frac{Y_B L}{q_{in}} = \frac{L^{16/13} \left(\frac{n}{Cu}\right)^{6/13}}{q_{in}^{7/13}} \quad \ldots\ldots (21)$$

$$Z_B = \frac{k\, L^{16a/13} \left(\frac{n}{Cu}\right)^{6a/13}}{q_{in}^{7a/13}} \quad \ldots\ldots (22)$$

And from Eq. 15

$$DU = \frac{z_{n\,B}^*}{t_{co\,B}^*} K_B^* = \frac{z'_{n\,B}}{t_{co\,B}^*} \quad \ldots\ldots (23)$$

in which $z'_{n\,B} \equiv z_n / Y = z_{n\,B}^* K_B^*$. Since $z_{n\,B}^* = (\tau_{nB}^*)^a$, the independent variable K^* can be replaced in the list of independent dimensionless variables affecting

DU by the new independent variable τ_n^*, since $K^* = z'_n/\tau_n^{*a}$. This replacement has two advantages. First, τ_n^* and z_n^* (or τ_n and z_n) have more physical significance than K^* and z_n^* (or k and z_n) in relation to surface irrigation. Further, by replacing the characteristic depth of infiltration Z_B by Y_B, the parameter a (and Eq. 22) is removed from the calculation of reference variables. Thus, the solutions for DU with different values of a can be compared without recalculating the reference variables and a sensitivity analysis on a can be made by direct observation of the solution curves.

Advance and recession curves for this dimensionless representation could be obtained for discrete values of K_B^*, $t_{co\,B}^*$, and a, with $V = D^* = q_{in}^* = L^* = 1$. Then, the solution for DU could be displayed in terms of $z'_{n\,B}$, $\tau_{n\,B}^*$ and a. Computationally, there is one drawback to this dimensionless solution. The solution technique for solving the zero-inertia equations uses discrete time steps for the computation of distance increments during advance. Advance is considered complete when $x^* > L^*$. Obtaining a solution that results in an L^* value of exactly one is unlikely. Thus, it would be more precise to compute the solution with $t_{co}^* = 1$ and transform the results, considered as follows.

For a given dimensional set of values for q, t_{co}, k, a, L, n, and z_n, there is one and only one value of DU. Thus, once the value for DU is determined from the solution of the governing equations, it can be displayed in any form as long as DU is the same for that same set of dimensional values. Since the relationships between the reference variables must be maintained, Eqs. 5–7 and 15 must also still hold. Thus, an indirect solution can be obtained. The possible choices available for reference variables X, Y, Z, Q, T (two of which can be fixed) are L, z_n, q_{in}, t_{co}, and τ_n. Two of these combination have been mentioned earlier, namely $Q_A = q_{in}$, $T_A = t_{co}$ and $Q_B = q_{in}$, $X_B = L$. Out of the remaining combinations, only $Y_C = z_n$, $T_C = \tau_n$ is of practical significance. This dimensionless system has been denoted by the subscript C. Specifying $T_C = \tau_n$ also gives $Z_C = z_n$ since $Z = z(T)$. Thus, the dimensionless system is also characterized by $z_n^* = 1$, $\tau_n^* = 1$ and $K^* = 1$. Solving for the reference variables from Eqs. 5–7 gives

$$Q_C = \frac{z_n^{16/9}}{\left(\frac{n}{Cu}\right)^{2/3} \tau_n^{1/3}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (24)$$

$$X_C = \frac{Q_c \tau_n}{z_n} = \frac{\tau_n^{2/3} z_n^{7/9}}{\left(\frac{n}{Cu}\right)^{2/3}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (25)$$

and from Eq. 15

$$DU = \frac{L_C^*}{q_{in\,C}^* t_{co\,C}^*} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (26)$$

The solution can be presented in terms of $q_{in\,C}^*$, L_C^* and a. Also, as with the previous (B) dimensionless system, a is not involved in the calculation of the reference variables.

Dimensionless Solutions

The governing equations for flow in a rectangular level basin (Eqs. 3, 4, and 8–12) were solved for the postirrigation profile of infiltrated water with $V^* = D^* = q^*_{in} = t^*_{co} = 1$ for $a = 0.5$, K^* ranging from 0.1–8, and L^* ranging from 0.05–5 (depending on K^*). The distribution uniformity was computed with Eq. 19 for each K^*, L^* combination and the resulting value of z'_n. A set of L^* values pertaining to given values of DU, i.e., 95, 90 ... 50, was then determined for each value of K^* with interpolating polynomials. These results were then transformed to represent solutions in terms of the two dimensionless systems considered earlier, namely $z^*_{nB} = \tau^*_{nB} = 1$ and $q^*_{inC} = D^*_C = 1$. The transformation was made point by point by using Eqs. 16–26 with all dimensional variables held constant. To simplify the computations, equations, were developed relating dimensionless values of one system to those of another. Curves through these points were then smoothed to yield the curves presented in Figs. 2 and 3. Fig. 2 can be used to determine the effect of the infiltration function on the distribution uniformity. The axes in Fig. 2 are made dimensional by multiplying values along the axes by Y_B and T_B from Eqs. 20 and 21 (similarly for Fig. 3 with Q_C and X_C from Eqs. 24 and 25). Fig. 3 can be used to determine the effect of unit flow rate and basin length on distribution uniformity. Fig. 3 resembles the level-basin design curves presented by the SCS in their border irrigation handbook (21). Unlike the SCS graphs, these curves allow for continuous, rather than discrete, variations in Manning roughness, net infiltration depth, and the corresponding required opportunity time. Both dimensionless representations can be used for evaluating existing systems, for designing new level-basin systems, and for developing management aids for actual operation. These curves represent a solution for an a value of 0.5 only. A complete solution to the design and management problem would include values of a ranging from 0–1.0.

Examples

For these examples, it is assumed that we want to apply a given depth of water everywhere in the field at some desired uniformity.

Design

1. Net depth: 10.16 cm (4 in.).
2. Infiltration time: 160 min.
3. Infiltration exponent: 0.5.
4. Roughness: 0.15.
5. Design uniformity: 80%.
6. Available flow rate: 283 L/s (10 cfs).
7. Field length: 268 m or 134 m (880 ft or 440 ft).
8. From Eqs. 24 and 25, calculate Q_C and X_C, with all units in meters and seconds:

$$Q_C = \frac{z_n^{16/9}}{\left(\frac{n}{Cu}\right)^{2/3} (\tau_n)^{1/3}} = \frac{(0.1016)^{16/9}}{(0.15)^{2/3}\,(9{,}600)^{1/3}}$$

$$= 0.00286\ \text{m}^3/\text{s/m} \quad \text{or} \quad 2.86\ \text{L/s/m}\ (0.0308\ \text{cfs/ft}) \ldots\ldots\ldots\ldots \quad (27)$$

$$X_C = \frac{Q_C \tau_n}{z_n} = \frac{(0.0029)(9{,}600)}{(0.1016)} = 270 \text{ m (887 ft)} \quad \ldots\ldots\ldots\ldots\ldots (28)$$

For a field length of 268 m, calculate $L_C^* = 268/270 = 0.993$. From Fig. 3 for $L_C^* = 0.993$, $DU = 80\%$, find $q_{inC}^* = 4.2$. The unit flow rate is $q_{in} = q_{inC}^* Q_C = 12.0$ L/s-m (0.129 cfs/ft). The border width is $B = 283/12.0 = 23.6$ m (77 ft). The area of the basin is 0.63 ha (1.56 acre).

For a field length of 134 m, $L_C^* = 0.496$. From Fig. 3, with $L_C^* = 0.496$, $DU = 80\%$, find $q_{inC}^* = 0.75$. The unit flow rate is $q_{in} = (0.75)(2.86) = 2.15$ L/s/m (0.0234 cfs/ft). The basin width is $B = 283/2.15 = 132$ m (433 ft). The area is 1.77 ha (4.4 acre).

This example is summarized in Table 1. By halfing the length of run for this example, the basin area can be almost tripled without any sacrifice in uniformity. Also, by decreasing the length of run, the distribution uniformity can be greatly improved for the same basin area, thus potentially decreasing the amount of water applied.

TABLE 1.—Design of Basin Dimensions

z_n, in centimeters (1)	τ_n, in minutes (2)	n (3)	Q_C, in liters per second-meter (4)	X_C, in meters (5)	L, in meters (6)	L_C^* (7)	DU, as a percentage (8)	q_{inC}^* (9)	q_{in}, in liters per second-meter (10)	Q_{in}, in liters per second (11)	B, in meters (12)	A, in hectares (13)
10.16	160	0.15	2.86	270	268	0.993	80	4.2	12.0	283	23.6	0.63
					134	0.496	80	0.75	2.15	283	132	1.77
							90	1.90	5.43	283	52.1	0.70

Fig. 2 can be used to estimate the effects of application depth for the two basin lengths. From Eqs. 20 and 21, calculate Y_B and T_B (from results in Table 1 with $L = 268$ m) with units in seconds and meters

$$Y_B = q_{in}^{6/13} L^{3/13} \left(\frac{n}{Cu}\right)^{6/13} = (0.012)^{6/13} (268)^{3/13} (0.15)^{6/13}$$

$$= 0.197 \text{ m (0.645 ft)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

$$T_B = \frac{L Y_B}{q_{in}} = \frac{(268)(0.197)}{(0.012)} = 4{,}390 \text{ sec} = 73.2 \text{ min} \quad \ldots\ldots\ldots\ldots\ldots (30)$$

For z_n of 10.16 cm, $z'_{nB} = 0.1016/0.197 = 0.516$, $\tau_{nB}^* = 160/73.2 = 2.19$. From Fig. 2, this corresponds to a distribution uniformity of 80% (Figs. 2 and 3 give identical results). Similar calculations can be made for 12.70 cm and 7.62 cm net depths. These results and the results for the shorter basin are given in Table 2. For the shorter basin, the distribution uniformity does not respond as drastically to changes in flow rate.

Management

1. Basin length: 201 m (660 ft).
2. Basin width: 61 m (200 ft).
3. Roughness: 0.10.
4. Flow rate: 283 L/s (10 cfs) ± 20%.

The infiltration of water into the soil can change considerably over the season. It can also change with the soil moisture deficit. For this example, the following

TABLE 2.—Comparison of Basin Designs

L, in meters (1)	q_{in}, in liters per second-meter (2)	n (3)	Y_B, in meters (4)	T_B, in minutes (5)	z_n, in centimeters (6)	z'_{nB} (7)	τ_n, in minutes (8)	τ^*_{nB} (9)	DU (10)
268	12.0	0.15	0.197	73.2	12.70	0.645	250	3.42	89
					10.16	0.516	160	2.19	80
					7.62	0.387	90	1.23	63
134	2.15	0.15	0.0758	78.7	12.70	1.68	250	3.18	86
					10.16	1.34	160	2.03	80
					7.62	1.01	90	1.14	69
134	5.43	0.15	0.116	47.8	12.70	1.09	250	5.23	93
					10.16	0.876	160	3.35	90
					7.62	0.657	90	1.88	80

TABLE 3.—Effects of Flow Rate and Net Depth on Distribution Uniformity and Application Time for Level-Basin Management

Q_{in}, in liters per second (1)	Net Depth		
	5.08 cm (2)	7.62 cm (3)	10.16 cm (4)
(*a*) Distribution Uniformity, as a percentage			
227	80	88	90
283	82	90	91
340	83	91	92
(*b*) Application Time, in minutes, rounded			
227	57	78	102
283	45	61	81
340	37	50	67

relationship is assumed for z_n and τ_n: 10.16 cm (4 in.), 300 min; 7.62 cm (3 in.), 240 min; 5.08 cm (2 in.), 160 min; all with $a = 0.5$. From Eqs. 20 and 21, $Y_B = 0.0985$ m (0.323 ft), $T_B = 71.1$ min. Values of z'_{nB} and τ^*_{nB} were calculated and distribution uniformities found from Fig. 2 for the three infiltration functions and for flow rates of 227, 283, and 340 L/s (8 cfs, 10 cfs, 12 cfs).

The results are given in Table 3. Table 3(*a*) shows the uniformities for 227 L/s, 283 L/s, and 340 L/s for the three infiltration functions previously given. Table 3(*b*) gives the resultant application or cutoff times. A table like this can be a very useful tool for the efficient application of irrigation water to level basins.

The values of distribution uniformity and application time in this table are estimates for the potential application efficiency. Factors not considered here or variation in the actual values of roughness, intake, and flow rate, could make these efficiencies unattainable. However, this makes the graphical solution presented here even more valuable, since the effects of the variability of these factors can be easily evaluated.

The final subsurface distribution of infiltrated water can be estimated by using dimensionless advance curves and a volume balance procedure, found in work by the first and second writers (5). Their solution and the one presented here differ in that in the former, recession is assumed to occur simultaneously over the entire field, whereas the curves presented here allows for recession to occur at the tail end of the basin first, progressing back to the head end.

Conclusion

The dimensionless solution of advance and recession in level basins was extended to show the distribution uniformities for a wide variety of conditions. This was then transformed into two representations of distribution uniformity that are more useful for designing and managing level basins. One solution displays the effects of net infiltration depth and the necessary infiltration opportunity time on distribution uniformity. The other displays the effects of field length and flow rate on distribution uniformity.

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Appendix II.—Notation

The following symbols are used in this paper:

A = basin area;
a = exponent for power infiltration function;
B = basin width;
C_u = units coefficient for Manning's n;
D = reference drag per unit plane area;
DU = distribution uniformity;
d = drag per unit plan area;

E_a = application efficiency;
f, f_1 = miscellaneous, unknown functions;
K^* = dimensionless relative infiltration constant;
k = constant for power infiltration function;
L = field length;
n = Manning roughness coefficient;
Q_{in} = flow rate of irrigation stream entering basin;
q_{in} = flow rate per unit width of irrigation stream entering basin;
S_o = field slope;
T = reference time;
t = time;
t_{co} = time at which irrigation stream is cut off;
V^* = dimensionless relative volume of water;
X = reference horizontal distance;
x = horizontal distance along basin;
x_a = advance distance along basin;
Y = reference surface water depth;
y = surface water depth;
Z = reference depth of water infiltrated;
z = depth of water infiltrated;
z_d = soil moisture deficit;
z_n = minimum or net depth of water infiltrated;
τ_n = infiltration opportunity time for depth z_n; and
ϕ, ψ = miscellaneous, unknown functions.

Subscripts

A = dimensionless system defined by $Q = q_{in}$, $T = t_{co}$;
B = dimensionless system defined by $Q = q_{in}$, $X = L$; and
C = dimensionless system defined by $Z = z_n$, $T = \tau_n$.

Superscripts

$*$ = indicated dimensionless variable.

SIMPLIFIED METHOD FOR RAINFALL INTENSITIES

By George H. Hargreaves,[1] F. ASCE

INTRODUCTION

In developing countries, rainfall intensity measurements are frequently not available, and, even when reported, deficiencies in record keeping, reading of charts and graphs, or conversions to other units may make the records less usable or less reliable than desired. If good universal relationships can be found between recorded data and intensity, probability distributions of rainfall intensity-duration-frequency amounts may then be found that will be useful for evaluation of records and for many purposes related to water management planning, design, and research.

The frequency of high-intensity precipitation is required for several engineering purposes, including agricultural drainage; estimation of extreme floods for inadequately gaged streams; estimation of erosion and reservoir sedimentation; and designing the dimensions of storm sewers, flood channels, and spillways.

This study explores the usefulness of some rainfall amount-intensity relationships. Those at least partially evaluated include regressions predicting maximum monthly rainfall from mean monthly rainfall, 10-yr return period, 24-h duration amounts from maximum monthly rainfall, and the full range of rainfall intensity-duration-frequencies from the 10-yr-24-h values.

Data from Central America, the United States, India, Australia, and, to a limited degree, the world were used. Additional study and local evaluation or verification is recommended prior to use of the methods presented, particularly in areas having rainfall regimes significantly different from those evaluated.

Maximum Monthly Precipitation, PMX.—The World Meteorological Organization, WMO, (8) gives 30-yr mean rainfall, PM, and 30-yr maximum for monthly values, PMX, for many weather stations. Hargreaves (4) used PM and PMX from WMO (8) to make a world water balance study. Since mean precipitation values are published more frequently than 30-yr maximum values, it was considered worthwhile to develop equations for estimating PMX from PM. The regression equation is in the form:

$$PMX = a + b\,(PM) \quad \ldots \quad (1)$$

[1]Res. Engr., Dept. of Agr. and Irrigation Engrg., Utah State Univ., Logan, Utah.

Note.—Discussion open until February 1, 1982. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on December 10, 1980. This paper is part of the Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. IR3, September, 1981. ISSN 0044-7978/81/0003-0281/$01.00.

TABLE 1.—Regression Coefficients for Estimating 30-yr Maximum Monthly Rainfall from Mean Monthly Rainfall

Country (1)	Regression coefficients		Coefficient of determination R^2 (4)	Standard deviation S (5)	Degrees of freedom DOF (6)
	a (2)	b (3)			
Albania	121	1.70	64	68	22
Argentina	109	1.52	42	60	66
Austria	26	2.01	83	40	53
Bangladesh	241	1.29	55	145	30
Brazil	146	1.27	70	78	63
Belgium	24	2.63	43	29	10
Bulgaria	7	2.22	75	28	14
Canada	18	1.97	64	35	36
Chad	46	1.49	71	69	31
Congo Republic	82	1.45	49	66	50
Czechoslovakia	19	1.93	72	24	34
Dahomey	95	1.31	70	43	25
Denmark	31	1.78	85	22	24
Dominican Republic	74	2.16	59	111	13
Ecuador	−14	2.17	85	65	11
France	16	2.38	78	138	103
Germany	16	2.12	77	30	103
Ghana	21	1.96	78	60	18
Greece	19	2.25	87	28	11
Hungary	48	1.71	45	28	17
Iceland	16	2.21	89	33	15
India	155	1.73	94	187	109
Ireland	−6	1.97	86	27	10
Italy	43	2.11	54	56	98
Ivory Coast	94	1.69	91	77	18
Japan	97	1.69	54	139	104
Korea	−35	3.04	88	79	31
Malagasy	95	2.19	72	149	60
Mali	78	1.50	78	64	35
Mauritius	5	2.22	72	99	32
Mexico	112	1.87	38	176	32
Mozambique	16	2.75	80	95	16
Netherlands	−2	2.31	81	28	9
New Zealand	91	1.59	70	60	60
Niger	−70	2.39	90	45	8
Nigeria	112	1.28	78	41	29
Norway	−11	2.51	72	34	13
Pakistan	71	2.08	97	42	10
Philippines	40	2.11	75	148	83
Portugal	18	2.70	77	96	68
Romania	43	1.70	69	24	26
Senegal	−2	1.61	73	119	14
Sierra Leone	−21	1.75	92	121	15
South Africa	109	1.50	31	57	33

TABLE 1.—*Continued*

(1)	(2)	(3)	(4)	(5)	(6)
Spain	5	2.96	74	48	33
Sri Lanka	158	1.79	74	97	19
Sudan	131	1.11	45	45	28
Sweden	13	2.13	69	23	39
Switzerland	74	1.75	77	65	30
Taiwan	−43	3.21	86	169	32
Tanzania	91	1.57	82	47	10
Thailand	159	1.17	66	52	27
Togo	113	1.32	66	48	26
Turkey	27	2.25	89	57	82
Uganda	136	1.13	66	54	14
United Kingdom	−23	2.47	89	73	110
United States	76	1.75	78	93	306
Upper Volta	28	1.71	95	32	7
Uruguay	−178	5.13	54	97	21
Yugoslovia	13	2.12	87	16	11
Zambia	129	1.26	72	48	17

For each country's data evaluated, values were recorded of a and b for PMX and PM in millimeters as well as the coefficient of determination, R^2; the standard deviation, S; and the degrees of freedom, DOF. The calculated regressions appear in Table 1.

For 69% of the countries shown, the regression equation predicts 70% or more in the variance in PMX. For those countries that have low values of R^2, it is assumed that two or more distinct types of rainfall or rainfall distributions were probably analyzed together.

The computer program printed the actual values and the computed values of PMX for those locations for which there is a large residual or a large difference between measured and estimated values. The large positive residuals, RPMX, were used in a regression analyses with values of PMX estimated from the various regression equations shown above. The resultant equation is

$$\text{RPMX} = 156 + 1.14\,\text{PMX} \quad \ldots\ldots\ldots\

and P 10, 24 amounts shown in the atlas by Hershfield (5). The equation for precipitation amounts in millimeters is:

$$P\,10,24 = 22 + 0.30\,\mathrm{PMX} \quad \ldots\ldots (3)$$

with $R^2 = 84\%$, $S = 20$ and DOF = 46. Similar equations with approximately equal values were developed by computer analysis of data from El Salvador and Nicaragua. Limited data from Honduras indicates that Eq. 3 overestimates P 10,24 for that area. Honduras is very mountainous and the relationship appears less reliable in the lee of mountains and in areas surrounded by mountains.

If daily rainfall records are available, P 10,24 can be determined by probability analysis. If T is the recurrence interval, n is the number of years of record (the number of items in the annual series) and m is the rank of the items in the series—m being 1 for the largest daily rainfall, then T can be estimated from one of the following equations. (Source of Eqs. 4 and 5 is WMO No. 168 TP 80, 1970. Source of Eq. 6 is ASCE Manual 28, Hydrology Handbook, 1949.)

$$T = \frac{n+1}{m} \quad \ldots\ldots (4)$$

$$T = \frac{n+0.4}{m-0.3} \quad \ldots\ldots (5)$$

$$T = \frac{n}{m-0.5} \quad \ldots\ldots (6)$$

The frequency analyses may be carried out with either an annual series or a partial series. The annual series uses the largest event in each year. The partial series uses all values exceeding an arbitrary value regardless of the year of occurrence. Hershfield (5) shows a linear relationship between annual series and partial duration series with partial duration series results equal to 1.11 × the annual series.

Hershfield's (5) atlas relies on the Gumbel procedure for fitting data to the Fisher-Tippett type 1 distribution to determine rainfall amounts for longer return periods. This latter procedure depends upon functions of the sample size and the standard deviation.

Hershfield (5) indicates that probable errors in depth duration rainfall values presented in the atlas are in the order of 10–20%. Depth-duration rainfall amounts can not be easily or accurately extrapolated over large areas. Many rainfall records are site specific. The average 10-yr event can not be determined accurately from 20 yr of records. Confidence limits and probable sampling errors are estimated by assuming a particular form of frequency distribution. The tabulation presented as Table 1 showing values to be used in Eq. 1 indicates that the skew in different rainfall distributions varies greatly.

It would seem logical to assume that length of record, representativeness of the site and the skew in the distribution typical for that particular climate or area have more influence on the accuracy of results than the form of the assumed distribution. Bell (1) indicates that for 68% confidence limits, one standard deviation with 25 yr of record, the upper limit for an estimated 50-yr period is 220 yr and the lower limit 12 yr.

Fourth and Sixth Root Relationships.—In a study of rainfall intensities in Central America (Un Metodo Sencillo Para Calcular Intensidades de Precipitacion en Centroamerica by Kenneth J. Vogler and George H. Hargreaves, paper accepted for publication by Comite Regional de Recursas Hidraulicos, Apartado 718, Tegucigalpa, Honduras, 1980, 23 p.) it was found that depth duration amounts varied approximately as the fourth root of the time, t, and the sixth root of the return period, T. This is partially confirmed by an equation presented by Bernard (2) for rainfall intensity, i. The equation is:

$$i = \frac{C}{t^n} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which C is a coefficient depending for its value on locality and n is an exponent. Values of n are presented for that portion of the United States east of longitude 100° W. The range of values of n is 0.73–0.82. Eq. 7 can be written in terms of depth, D, as

$$D = C \times t^{(1.0-n)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Powell (7) suggests that D can be approximated from, t, time in hours and frequency, F. The equation is:

$$D = K(t \times F)^{0.25} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Narayana (6) presents a table showing maximum storm rainfall in an area in India for durations of 1, 2, 3, and 4 days and for various return periods. If the 10-yr, 1-day maximum amount is assumed to be correct, other amounts can be estimated using the equation:

$$D = KT^{1/6}t^{1/4} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

in which K is a constant, then for return periods of 5–100 yr, the values shown by Narayana (6) can be reproduced from Eq. 8. The average difference between predicted and published values is 5% and the maximum difference 11.2%.

Eq. 10 was applied to the values in Hershfield's (5) atlas for the United States. It was assumed that the atlas values of P 10,24 were correct and that other amount and frequency values could be estimated from the given P 10,24 for the range of T from 5–100 yr and t of from 1–24 h. Differences between the atlas values and estimated values using Eq. 10 averaged 6%. The average R^2 of the regressions was 86%.

Bell (1) used data from the United States, including Hawaii, Alaska and Puerto Rico; Australia; and South Africa, to show graphically the relationships between 10-yr, 1-h rainfall and 1 h rainfall for return periods of 1, 2, 5, 25, 50, and 100 yr. In each case, the relationship is linear and passes through the origin of a linear plot. If the 1 h rainfall depths for 5, 25, 50, and 100-yr return periods are estimated using $T^{1/6}$, the estimated values differ by an average of 2.1% from the graphs presented by Bell (1). Data from the United States are also used to show ratios of amounts from various durations to one-h rainfall amounts. The ratios are as follows:

Duration, t	5 min	15 min	30 min	1 h	2 h
Ratio	0.29	0.57	0.79	1.00	1.25

The 30-min and 2 h ratios have an average difference from those estimated from the 1 h values using $t^{1/4}$ of 5.6%.

Bell (1) compares depth-duration ratios for the United States, the U.S.S.R. and Australia. The ratios are very similar in magnitude and range for these three large geographic areas. This indicates that ratios may be much the same for world-wide conditions.

From the above discussion and evaluation it appears that if the 10-yr-24-h precipitation, P 10,24 can be reliably determined from an annual or partial series, then errors in estimating other duration-return period amounts for durations of 30 min–4 days and return periods of 5–100 yr will not be excessive when estimated using Eq. 10. Values so estimated are well within the limits of accuracy attainable, considering limitations of sample size, accuracy, and representativeness of precipitation data. Eq. 10 has an advantage over most other distributions because of its simplicity and ease of computation.

Application and Procedures

World-wide coverage of mean monthly rainfall is presented by Wernstedt (8). The data are also available on computer magnetic tape, (Climatic Data Press, P.O. Box 413, Lemont, Pa. 16851) which makes data manipulation very easy. Eq. 1, for example, estimates 30-yr maximum monthly rainfall values from mean monthly rainfall. Thirty-year maximum values, for less complete coverage, are available from various sources including WMO (3) and Hargreaves (4). Eqs. 1 and 2 indicate that the values of the 30-yr maximum rainfall for locations without suitable records can be estimated from mean monthly rainfall. A method is also given for the envelope relationship. Eq. 3 estimates the 10-yr, 24-h rainfall from the 30-yr maximum, and Eq. 10 estimates the other duration-return period values for durations of 30 min to 4 days and return periods of 5–100 yr.

In developing countries, where rainfall intensity data are inadequate or not available, the above procedures provide a means of estimating values that can be used for planning and design of water resource developments. A major deficiency or weakness of the method is in the procedure for estimating 30 yr maximum values from mean monthly rainfall. The coefficients in the equation and the percentage of variation predicted are quite variable. Also, a 30-yr record may not contain a maximum value nearly as large as an average 30-yr return period amount or may include a maximum that is equivalent to an average return period of several hundred years. By comparing records from several locations, however, an attempt can be made to either eliminate or compensate for anomalous storms.

Summary and Conclusions

Maximum monthly precipitation can be reliably estimated from mean monthly rainfall. Outlying and envelope values can also be estimated. A fairly good relationship exists between 10-yr, 24-h rainfall and maximum monthly rainfall values. For durations of 30 min–4 days and return periods of 5–100 yr, precipitation amounts vary approximately as the fourth root of the time and the sixth root of the return period. These relationships apply over a wide range of climatic

regions and possibly are global in application. Additional study and statistical analyses are recommended in order to more clearly define reliability, universality, and general usefulness of the methods presented.

Care must be exercised in identifying unusual rainfall events and in compensating for them in the analyses. Short records and inadequate station coverage or nonrepresentative locations can lead to substantial error. Errors from short records and nonrepresentative data are frequently of sufficient magnitude so that accuracy of estimates will not be significantly influenced by the type of distribution used in analyzing the data. Additional statistical study is recommended in order to determine how well the equations presented herein and other similar ones can generally be applied to world-wide distributions.

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Appendix II.—Notation

The following symbols are used in this paper:

a = a regression coefficient;
b = a regression coefficient;
C = a coefficient depending on locality;
D = depth of precipitation or amount;
DOF = degrees of freedom (number of records minus 2);
F = frequency;
i = intensity (amount per unit time);
K = a constant for each location rainfall station;
m = rank of item in the series = 1 for the largest;
n = number of years of record or number of items in the series;
n = an exponent in the Eq. $i = C/t^n$;
PM = mean monthly precipitation or rainfall;

PMX = maximum monthly value of a 30 year series of precipitation measurements;
P 10,24 = 10 year return period 24 hour duration rainfall;
R^2 = percent of variance predicted by the regression equation;
S = standard deviation;
t = period of concentration (usually in min or h); and
T = return period in years.

Optimal Design of Border Irrigation Systems

By J. Mohan Reddy[1] and Wayne Clyma[2]

Introduction

Border irrigation systems, both graded and level, are widely practiced methods of surface irrigation. Effective designs of border systems have frequently been based on arbitrary constraints and performance criteria. The farmer, as the owner of the farm, is interested in the highest net benefits from crop production. Depending upon the amount of water available, the cost of production and the value of the produce, the farmer may or may not irrigate the whole farm. He is not sure how much area to irrigate to obtain maximum benefits. Hence, a procedure to analyze a given situation and optimally design the irrigation system would facilitate effective on-farm water management. This paper presents a procedure for optimal design of border irrigation systems based on maximization of profit while incorporating system operation constraints and the variables of the operating system.

Literature Review

Hall (4) presented a simple procedure to optimize the design of border irrigation systems. But only maximization of the application efficiency was considered. Vierhout (12) applied differential calculus to the optimal design of border and furrow irrigation systems. Once again, the criteria was to maximize application efficiency. Wu and Liang (13) presented a procedure to optimize only the length of run of a furrow irrigation system. There are many other variables, such as inflow rate, time of irrigation, and net depth of application, which should be simultaneously included in the optimization to arrive at an optimal design for the system.

Marjai (7) developed a procedure to determine the optimum inflow rate into a border with the other variables remaining constant. The objective was to maximize uniformity along the length of the border. Karmeli (6) presented a procedure to optimize the irrigation quality parameters, such as tailwater ratio,

[1]Postdoctoral Fellow, Dept. of Civ. Engrg.; Colorado State Univ., Fort Collins, Colo.
[2]Assoc. Prof., Dept. of Agr. and Chemical Engrg., Colorado State Univ., Fort Collins, Colo.

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deep percolation ratio, and water requirement efficiency for furrow irrigation. Though it is not difficult to extend the same procedure to border irrigation, the procedure becomes highly tedious and lengthy as the number of combinations of the variables increases. Besides, this procedure does not allow for system constraints. Similarly, Peri, Hart, and Norum (1979) developed the concept of optimal irrigation depth, considering the economic loss due to deficit and excess irrigation along with the distribution uniformity of the applied water. This procedure does not specify the optimal size of the irrigation unit, and the operational variables, such as the flow rate and the time of inflow, to achieve the desired performance. Recently, Reddy and Clyma (9) presented a procedure to optimize furrow irrigation system design based on minimum costs, design variables, performance parameters, and system constraints. A similar approach for borders is presented here considering maximization of profit after deriving the relationship between the design variables and the quality parameters.

System Relationships by Simulation

In irrigated agriculture maximum profit is obtained when the losses are minimum and the water requirement efficiency is at an optimum. Water requirement efficiency is defined as a percent of the amount of water made available for plant use to the water requirement at the time of irrigation (5). Both can be obtained simultaneously with proper design and appropriate management of the system. If losses are high, excess costs are incurred in providing irrigation water. Net benefits from crop production are reduced if the water requirement met by each irrigation is less than optimum. Thus, there is a trade-off between maximizing water requirement efficiency and minimizing losses. Therefore, the irrigation system should be designed for optimum net benefits.

A relationship between yield and design variables must be established to optimize the system design. This was achieved by a two-step process. First, a relationship was obtained between water requirement efficiency and the design variables using a hydraulic model. Second, a relationship between yield and water requirement efficiency was obtained using a crop production model and the hydraulic model. Later, these two relationships were combined with a mathematical programming technique to optimize the design of freely draining graded border and level basin irrigation systems.

Performance and Design Variables.—Hydraulic simulation of the applied water is an important component of this optimization model. Conceptually the hydraulic model represents accurately the operational conditions of the irrigation system. Actually the hydraulic model provides the volumes (depth) of water that enter the root zone, go to deep percolation, and run off the field. The hydraulic model simulates these volumes for given conditions of intake, slope, and design depth (irrigation requirement) and different combinations of the design variables, such as length of run, unit inflow rate, and time of irrigation.

All the system variables can be constrained to specified limits for given field conditions. The constraints were as follows:

$$Q_u \leq Q_{u,\max} \quad \ldots \ldots \ldots

$$T_i \leq T_{\max} \quad \ldots \quad (1c)$$

$$n_l L = L_F \quad \ldots \quad (1d)$$

in which Q_u = unit inflow rate into the border, L/s; T_i = time of inflow into the border, min; L = length of irrigation run, m; $Q_{u,\max}$ = maximum non-erosive stream size, L/s; $Q_{u,\min}$ = minimum flow rate required, L/s; $T_{\max}$ = maximum time available per irrigation, in minutes; L_F = length of the field, m; and n_l = number of lengths of run. After defining these limits, the values of the variables were discretized to a finite number and simulated by using the appropriate hydraulic model. The models used in this study were those of Strelkoff and Katopodes (11), and Clemmens and Strelkoff (1). Finer discretizations increase the cost of simulation. The length variable was discretized into a limited number because there are a limited number of acceptable alternatives, i.e., the length of run may be halved or reduced to one-third. For each combination of the variables, the water requirement efficiency, the volume of runoff, and the deep percolation volume were calculated. Here, only freely draining graded borders, and level basins are considered.

Graded Borders.—Graded borders are well suited to soils of moderate intake characteristics and slopes. Efficient irrigation is possible by balancing the advance and recession of water. In graded borders, the water frequently is freely draining at the downstream end of the field. Thus, runoff water becomes an important component of the irrigation system design. If desired, the detrimental effects of deep percolation may be incorporated into the problem.

After simulating for a set of given conditions and various combinations of the design variables—length of run, unit inflow rate, and time of irrigation—the values of the quality parameters, such as water requirement efficiency, and deep percolation and tailwater ratios were obtained. By a statistical analysis of the data, the following types of relationships were defined with a high degree of correlation, between the quality parameters and the design variables. They are

$$E_r = K_1 Q_u^{a_1} T_i^{b_1} L^{c_1} D_u^{d_1} \quad \ldots \quad (2)$$

$$R_t = K_2 Q_u^{a_2} T_i^{b_2} L^{c_2} D_u^{d_2} \quad \ldots \quad (3)$$

and $$R_p = K_3 Q_u^{a_3} T_i^{b_3} L^{c_3} D_u^{d_3} \quad \ldots \quad (4)$$

in which E_r = water requirement efficiency, in percent; R_t = tailwater ratio or the volume of runoff divided by the total volume applied; R_p = deep percolation ratio or the volume of deep percolation divided by the total volume applied; D_u = water requirement at the time of irrigation, in millimeters; and K_1–d_3 are constants that are site dependent. For given conditions and constraints, these equations provide the relationships between system performance and design variables for graded borders.

Level Borders.—A level border or basin is defined as an irrigation unit of zero slope with the downstream end diked. The tailwater ratio is zero because there is no runoff. Here, an approach similar to the one in the previous section is followed. Thus, the relationship is given as

$$E_r = K_4 Q_u^{a_4} T_i^{b_4} L^{c_4} D_u^{d_4} \quad \ldots \quad (5)$$

in which d_4 = a constant.

In level border irrigation the water requirement efficiency relationship is sufficient to describe the quality parameters. There is no runoff. Hence, deep percolation is the only loss in the field. Thus, the deep percolation ratio can be derived easily from the water requirement efficiency. The derivation is as follows:

$$V_p = K_5 Q_u T_l - K_6 L D_u \frac{E_r}{100} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

in which V_p = volume of deep percolation, in cubic meters; and K_5 and K_6 = units conversion factors. The deep percolation ratio is given as

$$R_p = \frac{V_p}{(K_5 Q_u T_l)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7a)$$

$$R_p = \frac{K_5 Q_u T_l - K_6 L D_u \dfrac{E_r}{100}}{K_5 Q_u T_l} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7b)$$

$$R_p = 1 - \frac{K_6 L D_u E_r}{100 K_5 Q_u T_l} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7c)$$

$$R_p = 1 - \frac{K_6 K_4 Q_u^{a_4} T_l^{b_4} L^{c_4} D_u^{d_4} L D_u}{100 K_5 Q_u T_l} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7d)$$

$$\text{or} \quad R_p = 1 - \frac{(K_6 K_4 Q_u^{a_4 - 1} T_l^{b_4 - 1} L^{c_4 + 1} D_u^{d_4 + 1})}{100 K_5} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Eqs. 5 and 8 provide the relationships between system performance and design variables for given conditions and constraints. Now, a relationship between system performance and yield is developed.

Yield and Performance.—Evaluation of the optimum level of crop production for a given irrigation system requires a crop production model that defines yield as a function of system performance at each irrigation during the season. The system performance at each irrigation in each section of the field is obtained from the hydraulic model. The model used in simulating the yield is presented elsewhere (10). The depth of water applied was assumed constant for each irrigation during the season. Any other sequence of depth of irrigations may be specified including empirical or experimental approaches to defining the depth or the sequence, or both. Once the optimal depth of irrigation and the crop production model are given, the relative yield of the crop as a function of different levels of water requirement efficiency, a constant at each irrigation during the season, was simulated. Different combinations of yield and water requirement efficiency were obtained by varying the design variables: inflow rate, length of run, and time of irrigation. Other variables such as slope, intake family, or design depth may be considered. A yield versus water requirement efficiency function can be developed from the above simulation data. For a given set of field conditions, the relationship of relative yield to water requirement as obtained from the simulation is shown in Fig. 1. The relationship was quadratic as follows:

$$Y_R = -0.279 + 0.0249E_r - 0.0001212E_r^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which Y_R = relative yield, and is defined as

$$Y_R = \frac{Y_a}{Y_{\max}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

in which Y_a = actual yield, kg/ha; and $Y_{\max}$ = potential yield under optimum

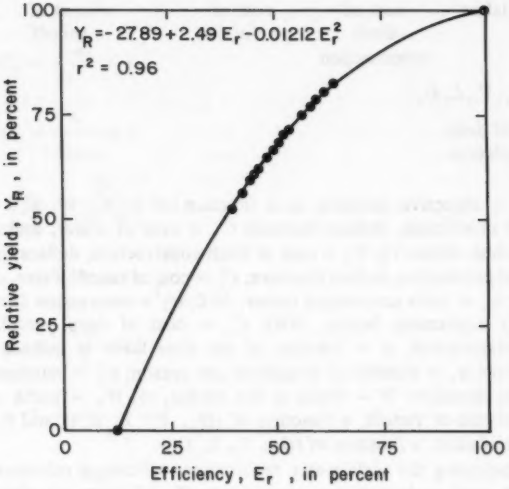

FIG. 1.—Relationship between Water Requirement Efficiency and Relative Yield of Wheat Crop

conditions, kg/ha. A high coefficient of correlation ($r^2 = 0.96$) was obtained between relative yield and the water requirement efficiency.

Problem Formulation

Problem formulation is an important component of any optimization. The problem is defined in terms of an objective function (either minimization or maximization) and related constraints. For this problem, the objective was to maximize the profit from crop production, i.e., searching for a particular value of water requirement efficiency and the corresponding values of the design variables that give the optimum net benefit. The profits were due to crop production in a particular field. The costs associated with irrigation system design are labor, water, energy, ditch construction, and any negative effects of runoff and deep percolation. If no direct costs of runoff or deep percolation can be quantified, then their costs are included in the increased amounts of water required. After the cost coefficients and the mathematical relationships

of the quality parameters are obtained, the problem can be formulated as shown below

$$\max G_o = P_c Y_R K_7 Ln_i Wn_w - C_1 \alpha_1 Q_u T_i Wn_i n_1 n_i$$

value of the produce; cost of water

$$- C_2 \alpha_2 \alpha T_i n_i n_w n_i - C_3 n_i W_F - C_4 K_7 Ln_i Wn_w - C_5 f_1(Q_u, T_i, L, D_u)$$

cost of labor; cost of ditch construction; cost of production; cost of runoff

$$- C_6 f_2(Q_u, T_i, L, D_u) \quad \ldots \quad (11)$$

cost of deep percolation

where G_o = objective function as a function of L, n_i, W, n_w, Q_u and T_i; P_c = profit coefficient, dollars/hectare; C_1 = cost of water, dollars/liter; C_2 = cost of labor, dollars/h; C_3 = cost of ditch construction, dollars/linear meter; C_4 = cost of production, dollars/hectare; C_5 = cost of runoff water, dollars/hectare-meter; α_1 = units conversion factor, 60.0; α_2 = conversion factor, 1/60.0; K_7 = units conversion factor, .0001; C_6 = cost of deep percolated water, dollars/hectare-meter; α = fraction of the time labor is utilized during the irrigation time; n_i = number of irrigations per season; n_w = number of borders in the width direction; W = width of the border, m; W_F = width of the field, m; f_1 = volume of runoff, a function of (Q_u, T_i, L, D_u); and f_2 = volume of deep percolation, a function of (Q_u, T_i, L, D_u).

After substituting the yield-water requirement efficiency relationship, Eq. 9, into Eq. 11, and neglecting the cost of runoff and deep percolated volumes the objective function becomes

$$\max G_o = P_c K_7 [-0.279 + 0.0249 E_r - 0.001212 E_r^2] Ln_i W n_w$$

value of the produce

$$- C_1 Q_u T_i Wn_i n_i n_w \alpha_1 - C_2 \alpha T_i n_i n_i n_w \alpha_2 - C_3 n_i W_F - C_4 K_7 Ln_i Wn_w \quad (12)$$

cost of water; cost of labor; cost of ditch construction; cost of production

By substituting the relationship between the design variables and the water requirement efficiency, Eq. 2, into Eq. 12, the objective function is given as:

$$\max G_o = P_c K_7 [-0.279 + 0.0249 K_1 Q_u^{a_1} T_i^{b_1} L^{c_1} D_u^{d_1} - 0.0001212 (K_1 Q_u^{a_1} T_i^{b_1} L^{c_1} D_u^{d_1})^2] Ln_i Wn_w$$

value of the produce

$$- C_1 Q_u T_i n_i Wn_w n_i \alpha_1 - C_2 \alpha T_i n_i n_w n_i \alpha_2 - C_3 n_i n_w W - C_4 K_7 n_i n_w LW \quad \ldots \quad (13)$$

cost of water; cost of labor; cost of ditch construction; cost of production

and the constraints, G_K, are given as:

$$K_1 Q_u^{a_1} T_i^{b_1} L^{c_1} \le 100 \rightarrow G_1 = K_1 Q_u^{a_1} T_i^{b_1} \frac{L^{c_1}}{100} \le 1 \quad \ldots\ldots\ldots\ldots (14a)$$

$$Q_u \le Q_{u,\max} \rightarrow G_2 = Q_u / Q_{u,\max} \le 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (14b)$$

$$Q_u \ge Q_{u,\min} \rightarrow G_3 = \frac{Q_{u,\min}}{Q_u} \le 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (14c)$$

$$L \ge L_{\min} \rightarrow G_4 = \frac{L_{\min}}{L} \le 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (14d)$$

$$L \le L_{\max} \rightarrow G_5 = \frac{L}{L_{\max}} \le 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (14e)$$

$$Q_u W = Q_F \rightarrow G_6 = \frac{Q_F}{(Q_u W)} \le 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (14f)$$

$$n_l n_w T_i \le T_{\max} \rightarrow G_7 = n_l n_w \frac{T_i}{T_{\max}} \le 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (14g)$$

$$n_l L = L_f \rightarrow G_8 = n_l \frac{L}{L_F} \le 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (14h)$$

$$n_w W \le W_F \rightarrow G_9 = n_w \frac{W}{W_F} \le 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (14i)$$

$$W \ge W_{\min} \rightarrow G_{10} = \frac{W_{\min}}{W} \le 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (14j)$$

$$W \le W_{\max} \rightarrow G_{11} = \frac{W}{W_{\max}} \le 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (14k)$$

in which $W_{\min}$ = minimum width of the border, m; $W_{\max}$ = maximum width of the border, m; $L_{\min}$ = minimum length of the run, m; $L_{\max}$ = maximum length of the run, m; and Q_F = total flow rate available at the farm, L/s.

In the above problem, the equality constraints were formulated as less than or equal to constraints, in conformity with the requirement for the geometric programming technique, which is explained in the next section. The constraint is so formulated that it is satisfied as an equality. In the solution of the problem, it is not assumed that the whole field is irrigated. Therefore, the constraint on the width of the field, Eq. 14*i*, is not specified as an equality. If it is profitable to the farmer, the solution will satisfy the constraint, Eq. 14*i*, as an equality and the farmer irrigates the whole field. Though, the less than or equal to constraint could have been specified on Eq. 14*h*, instead of Eq. 14*i*, for the present purposes, it is specified on the width constraint, assuming that the farmer irrigates the full length of the field, thereby adjusting the number of borders in the width direction if it is not profitable to irrigate the whole field.

Generalized geometric programming, which is applicable to engineering design problems of this type, is most appropriate for the above problem. The same technique was presented by Reddy and Clyma (9) for furrow irrigation systems. Generalized geometric programming, GGP, is formulated with an objective function of the form:

$$G_o(\bar{X})_{\min} = P_o(\bar{X}) - Q_o(\bar{X}) \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (15)$$

with K constraints of the form:

$$G_k(\bar{X}) = P_k(\bar{X}) - Q_k(\bar{X}) \leq 1.0, \; k = 1, 2, 3, \ldots K \quad \ldots \ldots \ldots \ldots \quad (16)$$

in which $\bar{X}$ = the vector of variables to be considered $(x_1, x_2, x_3, \ldots x_M)$; M = the number of variables; and P_k and Q_k = posynomial functions of the form:

$$P_k(\bar{X}) = \sum_{i=1}^{I} U_{ik}(\bar{X}); \quad k = 0, 1, 2, 3, \ldots K \quad \ldots \ldots \ldots \ldots \ldots \quad (17)$$

$$Q_k(\bar{X}) = \sum_{j=1}^{J} V_{jk}(\bar{X}); \quad k = 0, 1, 2, 3, \ldots K \quad \ldots \ldots \ldots \ldots \ldots \quad (18)$$

in which U_{ik} and V_{jk} = terms with positive and negative coefficients, respectively, in the objective function and the constraints; I = number of terms in the objective function or the constraints with positive coefficients $(P_k(\bar{X}))$; and J = number of terms in the objective function or the constraints with negative coefficients $(Q_k(\bar{X}))$. The terms U_{ik} and V_{jk} are defined as follows:

$$U_{ik} = C_{ik} \prod_{m=1}^{M} x_m^{\epsilon_{ikm}}; \quad k = 0, 1, 2, \ldots K \quad i = 1, 2, 3, \ldots I \quad \ldots \ldots \ldots \quad (19)$$

$$V_{jk} = C_{jk} \prod_{m=1}^{M} x_m^{\epsilon_{jkm}}; \quad k = 0, 1, 2, \ldots K \quad j = 1, 2, 3, \ldots J \quad \ldots \ldots \ldots \quad (20)$$

in which C_{ik} and C_{jk} = coefficients; ϵ_{ikm} and ϵ_{jkm} = exponents of the variables in the objective function and the constraints; and x_m = system variable.

Eqs. 15 and 16 are called signomials. A signomial is defined as the difference of two posynomials. In the formulation of GGP, the terms with the negative coefficients $Q_k(\bar{X})$ are moved to the right of the inequality, resulting in $P_k(\bar{X}) \leq 1 + Q_k(\bar{X})$. Dividing the inequality by the right-hand side term results in a rational function of posynomials. The posynomial in the denominator is reduced to a monomial (single term) by a process of condensation as defined by Dembo (2). A posynomial divided by a monomial is still a posynomial. This posynomial is then condensed to a monomial again by condensation. After the monomials are obtained, the constraints and the objective function are linearized by taking the natural logarithm of the monomial function. This set of equations is solved by linear programming. Convergence of the solution to the original problem is obtained by additional constraints called "cuts" to the original problem. By solving the linear program a finite number of times, an optimum solution is obtained to the original nonlinear problem. Being a nonlinear programming

problem, the global solution cannot be guaranteed. Different local optima are obtained by starting at different initial feasible solutions. The maximum of all the optima is considered the global solution to the problem. A signomial geometric programming code has been developed to solve the problem.

The values of the variables obtained from the above technique are continuous (non-integer). In the design of an irrigation system, some of the variables, such as the number of lengths of run, sets, and borders in the width direction, should have integer values. Therefore, a different technique was attached to the above procedure to obtain an optimal solution in terms of integers for the above variables. The branch-and-bound (3) technique was chosen to express the related variables in an integer form.

Optimal System Design

The generalized geometric programming technique presented above was applied to the optimal design of border irrigation systems: graded borders and level

TABLE 1.—Cost Coefficients, System Constants and System Constraints for Freely Draining Border

Parameters (1)	Value (2)
(*a*) Cost Coefficients	
Value of produce, dollars/m^3	56.8
Cost of production, dollars/hectare	494.0
Maximum production, cubic meters/hectare	17.41
Cost of labor, dollars/h	3.00
Cost of water, dollars/hectare-meter	40.0
Cost of ditch construction, dollars/linear meter	3.25
(*b*) System Constants	
Length of the field, m	805.0
Width of the field, m	402.0
Slope of the field, m/m	0.001
Roughness of the field, $m^{1/6}$	0.024
Depth of requirement, mm	76.0
Infiltration constants, $z = kt^a$	
k, mm/min^a	18.0
a	0.2716
Number of irrigations per season	5
α	1
(*c*) System Constraints	
$Q_{u,max}$, L/s	11.20
$Q_{u,min}$, L/s	0.92
Q_p, L/s	158.0
T_{max}, min	3,600.0
L_{min}, m	67.0
L_{max}, m	402.0
W_{min}, m	9.0
W_{max}, m	30.5

basins. The two cases are discussed separately as an example with specific conditions.

Example 1—Graded Border.—The data for this problem is presented in Table 1. Using this data the hydraulic model was simulated for different combinations of the design variables as presented in Table 2. A relationship of the following

TABLE 2.—Relationship between the Design Variables and the Water Requirement Efficiency for a Freely Draining Border

Inflow rate, in liters per second (1)	Time of inflow, in minutes (2)	Length of run, in meters (3)	Water requirement depth, in millimeters (4)	Water requirement efficiency, in percent (5)
3.28	40.0	91.50	51	99.50
3.28	60.00	91.50	76	76.33
3.28	80.00	91.50	76	82.74
3.28	100.00	91.50	102	65.00
3.28	150.00	91.50	76	98.22
4.16	30.00	91.50	51	96.40
4.16	60.00	91.50	76	77.29
4.16	80.00	91.50	76	83.58
4.16	100.00	91.50	102	65.50
4.16	150.00	91.50	76	98.93
4.62	70.00	183.00	102	59.40
5.58	60.00	183.00	58	99.50
5.58	80.00	183.00	76	83.55
5.58	100.00	183.00	76	88.87
5.58	150.00	183.00	102	73.10
7.41	40.00	183.00	76	69.88
7.41	60.00	183.00	76	78.31
7.41	80.00	183.00	102	62.70
7.41	100.00	183.00	76	89.90
7.41	150.00	183.00	76	99.97
4.62	80.00	366.00	102	66.70
4.62	100.00	366.00	76	83.68
4.62	120.00	366.00	76	89.37
4.62	150.00	366.00	76	96.05
4.62	180.00	366.00	76	99.88
6.04	70.00	366.00	76	76.51
6.04	90.00	366.00	144	56.10
6.04	120.00	366.00	76	91.85
6.04	150.00	366.00	102	72.60
7.41	100.00	366.00	102	66.00

form was obtained between water requirement efficiency and the design variables.

$$E_r = 2{,}045 Q_u^{0.07678} T_l^{0.28299} L^{-0.04829} D_u^{-0.9938} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (21)$$

A very good correlation ($r^2 = 0.99$) was obtained between the predicted and actual water requirement efficiency. A comparison of actual versus predicted water requirement efficiency is shown in Fig. 2.

After obtaining the relationships between the design variables and the irrigation quality parameters, the problem was formulated in terms of the cost coefficients, system constraints, system constants, and the design variables. In the present study, the effect of deep percolation and tail water are not considered. But, when appropriate cost coefficients are available, they can be incorporated into the optimal design process. For the given situation P_c was calculated by

$$P_c = Y_{\max}\left(\frac{\text{m}^3}{\text{ha}}\right) \times P_u \text{ dollars/cubic meter}$$

$$= 17.41\frac{\text{m}^3}{\text{ha}} \times \left(\frac{56.8\$}{\text{m}^3}\right) = 989 \text{ dollars/hectare} \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

in which P_u = price per unit of produce ($\$/\text{m}^3$). After substituting Eq. 22

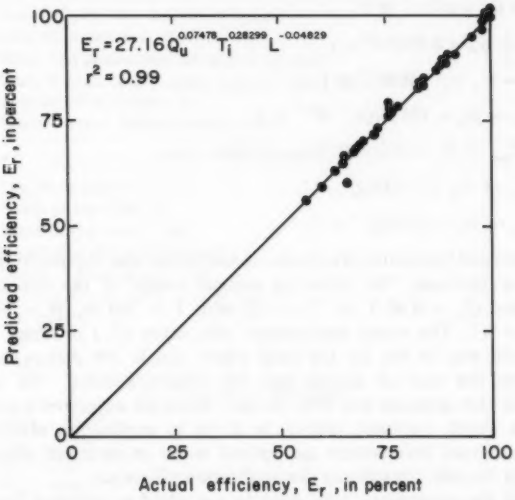

FIG. 2.—Actual Versus Predicted Water Requirement Efficiency for Freely Draining Border

and the cost coefficients into Eq. 11, and neglecting the coefficients of runoff and deep percolation, the objective function becomes:

$$\max G_o = \underbrace{989 Y_R K_7 L n_l W n_w}_{\text{value of produce}} - \underbrace{0.0012 Q_u T_i W n_l n_w}_{\text{cost of water}}$$

$$- \underbrace{0.25 T_i n_l n_w}_{\substack{\text{cost of} \\ \text{labor}}} - \underbrace{3.95 W n_l n_w}_{\substack{\text{cost of ditch} \\ \text{construction}}} - \underbrace{0.0494 L n_l W n_w}_{\substack{\text{cost of} \\ \text{production}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

By substituting Eqs. 9 and 21 (at $D_u = 76$ mm) into Eq. 23 and simplifying, the result becomes

$$\max G_o = -0.02758Ln_l Wn_w + 0.06688Q_u^{0.07478} T_l^{0.283} L^{0.9517} n_l Wn_w - 0.0884Q_u^{0.1496} T_l^{0.566} L^{0.9034} n_l Wn_w - 0.0012Q_u T_l Wn_l n_w - 0.25T_l n_l n_w - 3.25n_l Wn_w - 0.0494Ln_l Wn_w \quad (24)$$

The system constraints are given as follows:

$$E_r \le 100 \rightarrow G_1 = 0.2716Q_u^{0.07478} T_l^{0.28299} L^{-0.04829} \le 1 \quad (25a)$$

$$n_l L = L_F \rightarrow G_2 = 0.001242Ln_l \le 1 \quad (25b)$$

$$n_w W \le W_F \rightarrow G_3 = 0.002487Wn_w \le 1 \quad (25c)$$

$$L \le L_{\max} \rightarrow G_4 = 0.003727L \le 1 \quad (25d)$$

$$L \ge L_{\min} \rightarrow G_5 = 67L^{-1} \le 1 \quad (25e)$$

$$W \le W_{\max} \rightarrow G_6 = 0.0328W \le 1 \quad (25f)$$

$$W \ge W_{\min} \rightarrow G_7 = 9.146W^{-1} \le 1 \quad (25g)$$

$$Q_u W = Q_F \rightarrow G_8 = 151.42Q_u^{-1} W^{-1} \le 1 \quad (25h)$$

$$n_l n_w T_l \le T_{\max} \rightarrow G_9 = 0.0002778n_l n_w T_l \le 1 \quad (25i)$$

$$Q_u \le Q_{u,\max} \rightarrow G_{10} = 0.1350Q_u \le 1 \quad (25j)$$

$$Q_u \ge Q_{u,\min} \rightarrow G_{11} = 0.92Q_u^{-1} \le 1 \quad (25k)$$

The generalized geometric programming technique was applied to the solution of the above problem. The following optimal values of the design variables were obtained $Q_u = 4.98$ L/s; $T_l = 122$ min; $L = 269$ m; $W = 31$ m; $n_l = 3$; and $n_w = 13$. The water requirement efficiency (E_r) obtained was 91%. The net profit was \$9,833 for the field which equals 304 dollars/hectare. At the optimum, the cost of design was 186 dollars/hectare. The application efficiency for the optimum was 51%. In fact, when the objective is to maximize net benefits, much emphasis cannot be given to application efficiency. The technique presented here selects the optimal water requirement efficiency and yield without directly considering the application efficiency.

In defining the constraints, care must be exercised in selecting proper limits for the variables. These limits should not be different from the limits used in simulating the hydraulic model; if so, the relationships developed may not be valid. If a wide range of alternatives for the design variables are considered, these same limits should be included in the hydraulic simulation model.

Example 2—Level Basin.—The optimization technique was applied for a level basin. The system cost coefficients, constants, and constraints are given in Table 3. The hydraulic model was used to simulate the relationships between the quality parameters and the design variables. The different combinations of the design variables used in the simulation are presented in Table 4. For the given situation, the following relationships were obtained between the quality parameters and the design variables:

$$E_r = 3{,}103Q_u^{0.8232} T_l^{0.9182} L^{-0.8393} D_u^{-0.9201} \quad (26)$$

or $E_r = 85Q_u^{0.8232}\, T_i^{0.9182}\, L^{-0.8393},\ D_u = 50\ \text{mm}$ . . . . . (27)

and the equation for R_p is given as:

$$R_p = 1 - 31Q_u^{-0.1768}\, T_i^{-0.0818}\, L^{0.1607}\, D_u^{0.0799} \quad \text{. . . . . (28)}$$

or $R_p = 1 - 0.85Q_u^{-0.1768}\, T_i^{-0.0818}\, L^{0.1607},\ D_u = 50\ \text{mm}$ . . . . . (29)

A good correlation ($r^2 = 0.96$) between the actual and predicted water requirement

TABLE 3.—Cost Coefficients, System Constants, and System Constraints for a Level Basin Irrigation System

Parameter (1)	Value (2)
(*a*) Cost Coefficients	
Value of produce, dollars/kilogram	1.00
Cost of production, dollars/hectare	1,482.0
Maximum production, kilograms/hectare	2,500.0
Cost of ditch construction, dollars/linear meter	6.56
Cost of labor, dollars/h	3.0
Cost of water, dollars/hectare meter	40.0
(*b*) System Constants	
Length of the field, m	335
Width of the field, m	302
Slope of the field, m/m	0.0
Roughness of the field, $m^{1/6}$	0.15
Depth of requirement, mm	50
Infiltration constants, $z = kt^a + ct$	
k, mm/h^a	28.52
a	0.1088
c, mm/h	2.25
(*c*) System Constraints	
$Q_{u,\max}$, L/s	11.20
$Q_{u,\min}$, L/s	0.92
Q_F, L/s	52.0
$T_{\max}$, min	3,600.0
$L_{\max}$, m	168
$L_{\min}$, m	67
$W_{\max}$, m	30.5
$W_{\min}$, m	9

efficiency was obtained. A comparison of predicted versus actual water requirement efficiency is presented in Fig. 3. The above relationship was used to formulate the problem.

The profit coefficient for the given situation is calculated as:

$P_c = Y_{\max}$ (kilograms/hectare) $\times\ P_u$ (dollars/kilogram) $= 2{,}500 \times 1$

$= \$2{,}500/\text{ha}$ . . . . . (30)

After substituting the cost coefficients from Table 3 and Eq. 27 into Eq. 12

and simplifying, the objective function becomes:

$$\max G_o = -0.06973 L n_l W n_w + 0.529 Q_u^{0.8232} T_l^{0.9182} L^{0.1607} n_l W n_w - 0.2189 Q_u^{1.646} T_l^{1.836} L^{0.6786} n_l W n_w - 6.56 n_l W n_w - 0.1482 L n_l W n_w - 0.0012 Q_u T_l n_l n_w^{-0.25} T_l n_l n_w \quad (31)$$

and the system constraints are given as

$$E_R \le 100 \rightarrow G_1 = 0.85 Q_u^{0.8232} T_l^{0.9182} L^{-0.8393} \le 1 \quad (32a)$$

$$n_l L \le 335 \rightarrow G_2 = 0.00298 L n_l \le 1 \quad (32b)$$

$$n_w W \le 302 \rightarrow G_3 = 0.00331 n_w W \le 1 \quad (32c)$$

$$Q_u W \le Q_F \rightarrow G_4 = 0.01929 Q_u W \le 1 \quad (32d)$$

$$n_l n_w T_l \le T_{\max} \rightarrow G_5 = 0.00022 T_l n_l n_w \le 1 \quad (32e)$$

$$W \le 45.7 \rightarrow G_6 = 0.05263 W \le 1 \quad (32f)$$

$$W \ge 24.4 \rightarrow G_7 = 9.15 W^{-1} \le 1 \quad (32g)$$

TABLE 4.—Relationship between Design Variables and Water Requirement Efficiency for Level Basin Irrigation System

Length of run, in meters (1)	Inflow rate, in liters per second (2)	Time of inflow, in minutes (3)	Water requirement depth, in millimeters (4)	Water requirement efficiency, in percent (5)
67	1.00	50.00	50	88.50
67	1.00	50.00	76	59.00
67	1.87	25.00	58	68.00
67	1.87	25.00	76	52.00
67	1.87	25.00	89	44.50
67	1.87	40.00	76	86.00
67	1.00	50.00	114	39.30
67	1.87	56.00	102	91.00
67	1.87	56.00	114	81.00
100	1.00	120.00	76	88.90
134	3.76	25.00	76	58.00
134	3.74	33.00	76	69.00
134	3.74	45.00	76	97.00
134	4.66	30.00	114	53.00
134	3.76	50.00	76	100.00
168	1.87	120.00	102	74.70
168	3.74	40.00	102	52.00
201	2.79	50.00	76	66.00
201	3.74	65.00	76	92.00
201	4.66	45.00	127	49.00
201	4.66	50.00	102	66.00
268	2.79	100.00	76	80.00
268	3.74	80.00	76	87.00
268	4.66	60.00	76	79.00
268	5.58	50.00	89	68.00

$$Q_u \leq 11.2 \rightarrow G_8 = 0.0894 Q_u \leq 1 \quad \ldots \quad (32h)$$

$$Q_u \geq 0.918 \rightarrow G_9 = 0.918 Q_u^{-1} \leq 1 \quad \ldots \quad (32i)$$

$$L \leq 167 \rightarrow G_{10} = 0.005988 L \leq 1 \quad \ldots \quad (32j)$$

By applying the generalized geometric programming technique, the following optimal values of the design variables were obtained. $Q_u = 1.20$ L/s; $T_i = 110$ min; $L = 168$ m; $W = 43.3$ m; $n_t = 2$; and $n_w = 7$. The water requirement efficiency satisfied at the optimum was 100%. The maximum net profit at the optimum was \$5,797/per field or \$573/ha. The application efficiency found at the optimum was 93%.

In the analysis, the cost of runoff and deep percolation water was included indirectly in the cost of water provided in excess of the requirement in the root zone. But, the negative effects of these parameters (runoff and deep

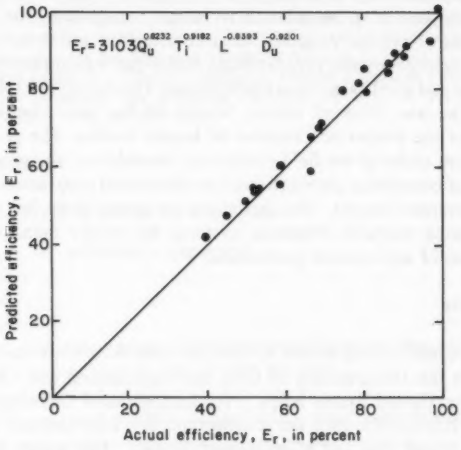

FIG. 3.—Actual Versus Predicted Water Requirement Efficiency for Level Border

percolation) were not considered because of lack of appropriate cost coefficients for these parameters. The cost coefficient for runoff includes the cost of removing the excess water from the field, and the negative effects on water quality. Similarly, the cost coefficient for deep percolation must take into account the effects of waterlogging and fertilizer leaching on crop yield. The cost coefficients should be given in terms of dollars/unit volume of water. Once these coefficients are available, they can be incorporated into the optimization process. This would help in evaluating different management practices in controlling the quality of irrigation return flow while increasing agricultural production.

An implicit assumption in the problem was that the farmer can apply the total available flow rate on one border. No consideration was given to irrigating more than one border at a time. This was not considered in the problem to

reduce the number of variables considered, but can be incorporated into the optimization process if desired. It was also assumed that the relationship between the yield and water requirement efficiency was identical under graded and level basin irrigation systems.

Summary and Conclusions

A combination approach of simulation and mathematical programming was used to develop an optimal system design. First, the irrigation quality parameters were developed in terms of the application system design variables using a hydraulic simulation model. Second, a relationship was developed between crop yield and the system quality parameters (water requirement efficiency) using a crop production function. Maximization of net benefits was the objective. The value of the produce, and the costs of labor, water, ditch construction, and crop production were considered in the objective function. The negative effects of runoff and deep percolation were not considered. The problem was defined in terms of system variables, cost coefficients and system constraints, and the generalized geometric programming technique was applied to the optimal design of border and level basin irrigation systems. The design variables considered were the inflow rate, time of inflow, length of the run, number of lengths of run, width of the border and number of border widths. The procedure gives an optimal design under given field conditions. In addition, the procedure shows the possibility of combining simulation and mathematical programming techniques in optimizing system designs. The technique presented provides guidelines for improving existing on-farm irrigation systems for better management of the scarce resources of agricultural production.

Acknowledgments

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Appendix II.—Notation

The following symbols are used in this paper:

a = infiltration exponent;
a_1, b_1, c_1, d_1
a_2, b_2, c_2, d_2
a_3, b_3, c_3, d_3
a_4, b_4, c_4, d_4 = exponents in irrigation quality parameter equations;
C_1 = cost of water, dollars per liter;
C_2 = cost of labor, dollars per hr;
C_3 = cost of ditch construction, dollars per linear meter;
C_4 = cost of production, dollars per hectare;
C_5 = cost of runoff water, dollars per hectare-meter;
C_6 = cost of deep percolation, dollars per hectare-meter;
c = constant in infiltration function;
D_u = water requirement at the time of irrigation, mm;
E_r = water requirement efficiency in percent;
G_o = objective function in terms of cost coefficients, profit coefficient and the system variables;
G_k = system constraints in terms of system variables;
I = number of terms in objective function or the constraints with positive coefficients;
J = number of terms in objective function or the constraints with negative coefficients;
K = number of constraints in problem;
K_1, K_2, K_3, K_4 = proportionality constants in irrigation quality parameter equations;

K_5 = units conversion factor, 0.06;
K_6 = units conversion factor, 0.001;
K_7 = units conversion factor, 10,000;
k = constant in infiltration function;
L = length of irrigation run, m;
L_F = length of field, m;
L_{max} = maximum length of run, m;
L_{min} = minimum length of run, m;
M = number of variables in the problem;
n_i = number of irrigations per season;
n_l = number of lengths of run;
n_w = number of borders in width direction;
P_c = profit coefficient, dollars/hectare;
P_k, Q_k = posynomial functions in objective function and the constraints;
P_u = unit price of produce, dollars/kilogram or m^3;
Q_F = total flow rate available at farm, L/s;
Q_u = unit inflow rate into border, L/s;
$Q_{u,max}$ = maximum non-erosive stream size into border, L/s;
$Q_{u,min}$ = minimum flow rate into border, L/s;
R_p = deep percolation ratio;
R_t = tail water ratio;
r^2 = correlation coefficient;
T_i = time of inflow into the border, in minutes;
T_{max} = maximum time available per irrigation, in minutes;
t = time variable, in minutes;
U_{ik}, V_{jk} = terms in the objective function and the constraints;
V_p = volume of deep percolation, m^3;
W = width of the border, m;
W_F = width of the firld, m;
W_{max} = maximum width of border, m;
W_{min} = minimum width of border, m;
x_m = system variable;
Y_a = actual yield, kilograms/hectare;
Y_R = relative yield;
Y_{max} = potential yield under optimum conditions, kilograms/hectare;
z = cumulative infiltration, mm;
α = fraction of the time labor is utilized during the irrigation time;
α_1 = units conversion factor, 60.0;
α_2 = units conversion factor, 1/60.0; and
$\epsilon_{ikm}, \epsilon_{jkm}$ = exponents of system variables in objective function and constraints.

HYDRAULIC TESTING OF PLASTIC FILTER FABRICS

By Yung Hai Chen,[1] M. ASCE, Daryl B. Simons,[2] F. ASCE
and Phillip M. Demery[3]

INTRODUCTION

Plastic filter fabrics are made from synthetic fibers. They are of two types, i.e., either a woven fabric featuring distinct open areas, or a nonwoven fabric that consists of a random fibrous mass of uniform thickness with torturous paths through the fabric. The fabric fiber consists predominantly of polyvinylidene chloride, nylon, polyester, or polypropylene yarns, filaments, or fibers. These synthetic yarns give rise to the term "plastic." However, the terms filter fabric and cloth filters are synonymous with plastic filter.

Filter fabrics, being both economical and durable, can be used as an alternative to a graded filter. Filter fabrics are economical because costs of both natural filter material and construction are increasing. Like granular filters, plastic fabric filters are designed to be highly permeable to water and yet constrain soil particles.

Plastic filter fabrics have been used to help protect coastal structures associated with rivers, lakes, canals, dams, and drainage systems of all types. Principally, the filters serve as bank protection, subdrainage protection, and as a means of protecting the foundation soil. These filters retain their strength undiminished by long exposure to salt or fresh water. Many successful applications of plastic filters have been documented, such as the usages of plastic filters to protect beaches in Florida (1), to control the erosion problems at bridges (4) by the Corps of Engineers, and to serve as a road support on a soft, spongy Alaskan soil (7).

An interaction exists between the plastic filter fabric characteristics, soil type used, and ground-water conditions. In analyzing the effectiveness of a filter fabric, the performance mechanism must be investigated considering the aforementioned variables. In establishing a conducive testing environment, it is possible to study the hydraulic properties of plastic filters associated with various soil types and water head. It is the intent of this study to provide information

[1]Assoc. Prof. of Civ. Engrg., Colorado State Univ., Fort Collins, Colo. 80523.
[2]Assoc. Dean for Research and Prof. of Civ. Engrg., Colorado State Univ., Fort Collins, Colo. 80523.
[3]Hydr. Engr., United States Dept. of Interior, Water and Power Resources Service, Denver, Colo.

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regarding nonwoven filter fabric performance in the hope of developing an adequate set of guidelines and criteria for plastic filter fabric selection, particularly for subsurface drainage systems where filter fabrics are predominantly of the nonwoven type.

Previous Laboratory Test and Selection of Filters

In selecting a plastic filter, the fabric permeability should be adequate to pass water from the soil while having a pore structure that at the same time will hold back soil. This rationale suggests that design specifications could be developed based on soil type, ground-water conditions, and filter cloth characteristics. Although current field applications show plastic filters have promise, hydraulic characteristics testing has so far failed to relate filter and soil characteristics with system performance.

The Corps of Engineers is one of the few agencies to document results of their hydraulic characteristics testing (2). Using soil types of known gradation, the Corps has established a set of guidelines pertaining to the use of plastic filters. Their results indicate that filter cloth selection is dependent on the D_{85} size of the soil, the EOS (equivalent opening size), and a term denoted as a gradient ratio. Two types of tests were conducted by the Corps: (1) Filtration tests; and (2) gradient tests.

Based upon analysis of the filtration tests, the Corps of Engineers recommended the following criteria be used in selecting a plastic filter. When the filter fabric is adjacent to granular materials containing 50% or less by weight of silt (material of little or no plasticity, passing the No. 200 sieve): (1) The fabric pore size estimated by EOS should be smaller than the D_{85} of the soil where the D_{85} is the diameter of the particle in which 85% by weight of the soil if finer; and (2) the open area of woven filters should not exceed 50%.

When the filter fabric is adjacent to soils having little or no cohesion containing more than 50% silt by weight: (1) EOS should be no larger than the opening in the United States Standard Sieve No. 70; and (2) open area of woven fabrics should not exceed 10%. To reduce the chance of clogging, no fabric should be specified with an EOS sieve number larger than the opening of a United States Standard Sieve No. 100 (0.0059 in., 0.15 mm). Filter fabrics require a sand blanket of thickness equal or larger than 6 in. to be placed between the filter and the soil for soils with 85% or more smaller than the No. 200 sieve.

The gradient ratio is a parameter the Corps of Engineers uses in filter comparison. This parameter is developed from permeameter testing and is a result of hydraulic gradient manipulation:

$$C = \frac{h_i}{h_t} \quad \ldots\ldots\ldots\

The design concept of the parameter is that as plugging occurs, there is a water pressure buildup immediately above the plastic filter. Some filter fabrics being tested developed a gradient ratio greater than 1.0 when subjected to a silty sand.

Rosen and Marks (7) studied the behavior of plastic filters at the University of Tennessee in 1975. Unlike the Corps of Engineers' filtration tests which were conducted for 60 h, their experiment was conducted 21 days–28 days or until the flow through the system became constant. Furthermore, the filtration and gradient ratio tests were conducted simultaneously. Twenty soil types were tested altogether, however, only one filter fabric was considered, i.e., Mirafi 140 (a nonwoven filter). The soil types were produced by adding various amounts of either silt, kaolinite, or montmorillonite to a river sand base soil.

Testing indicated a soil compaction occurred within the soil. This was deduced from permeability decreases as well as analysis of hydraulic gradients. The percentage of soil actually trapped in the fabric was quite small and did not vary significantly with soil type. The more soil was well graded, the smaller the amounts of fine particles that passed through the filter cloth. Rosen and Marks concluded that the design criteria of plastic filters must be the same as those established for conventional aggregate filters. The nonwoven filter tested was determined to be effective in subdrainage applications for a relatively wide range of soil conditions.

Ogink (6) investigated the characteristics of the fabric aperture and the blocking of fabrics by sand particles. Willardson and Walker (8) conducted tests to evaluate the response of using filter fabrics as drain envelope material to problem soils needing drainage. These writers, the Corps of Engineers, and other investigators have provided the public with some basic guidelines on filter cloth use. It is still the practice in some Corps districts not to use filter cloth within critical regions. The testing has opened the door for filter cloth applicability; however, it is time to alleviate the trial and error methodology of applying the filter cloths while at the same time, broadening the scope of the plastic filter fabric industry.

Laboratory Testing—Engineering Research Center, Colorado State University

Test Apparatus.—The experimental set-up (5) included a large head box, a small rectangular constant head box, 12 permeameters, a manometer board with tubes, weighing containers, recirculating pump, and drainage. Fig. 1(*a*) shows the flow diagram for these apparatus. This system allows for the testing of 12 filter systems simultaneously under identical hydraulic heads.

The large head box was constructed to intercept the city water and then convey this water to the small rectangular head box. This large wooden box was square with 8 ft (2.95 m) sides and 2 ft (0.61 m) depth. The purpose of the large head tank was two fold. The tank was fitted with a network of steamlines so that the water could be heated and maintained at a desired warmer temperature by using a thermostat. Also, the tank served as a stilling basin to decrease the inflow turbulence and air entrainment.

The small constant head box, 2 ft (0.61 m) in width, 4 ft (1.22 m) in length, and 2 ft (0.61 m) in depth, could be moved in a vertical direction along a support in order to vary the hydraulic head. Water supplied to each of the

12 permeameters via the 12 outlets was maintained at constant pressure head by means of the constant head box. Plastic tubing connected the head box outlet to the permeameters. The cylindrical permeameters were constructed of clear Lucite ®, 11.5 in. (292 mm) in diameter, with three distinct sections [see Fig. 1(*b*)]. The top section consisted of a gate valve to regulate incoming water flow and a distributor head to spread out the flow once the water entered the permeameter system. Standpipes were connected to the top of the permeameters to allow the air to escape before any serious accumulations developed. The middle section was used to contain a 4-in. (102-mm) deep soil column.

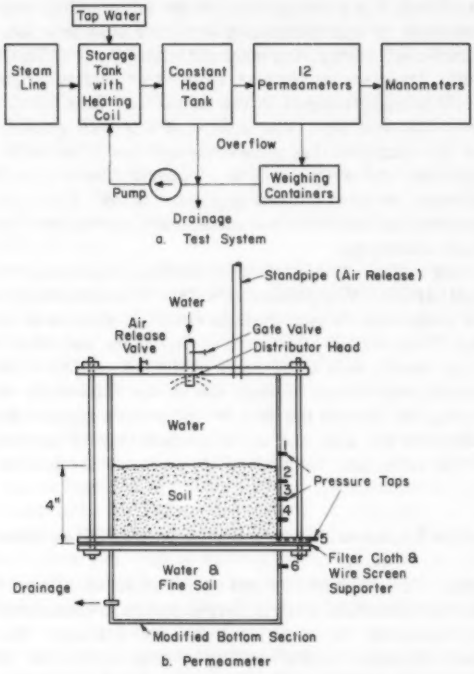

FIG. 1.—Laboratory Test System

The top and middle permeameter sections fit together flush with the plastic filter and screen support positioned directly below. A third section was attached to the bottom of the screen support to close the system from the surroundings. In order to measure the pressure head exerted on the soil column and the filter fabric, six pressure taps were installed on the vertical wall of each permeameter and connected to manometers. The first pressure tap was located 1 in. above the soil surface, whereas the second, third, and fourth pressure taps lay at 1 in. increments within the soil depth starting at the soil surface.

The fifth pressure tap was located 0.125 in. (3.2 mm) above the filter fabric and the sixth pressure tap was secured on the bottom section below the filter fabric.

A 5.0 hp pump was connected to the system to recirculate the water. It was thought that in recirculating the water, temperature fluctuations and air bubbles would be minimized. However, the pump was later removed to increase the fresh water supplied to the system in order to decrease the amount of bacterial activity within the system.

Test Procedure.—In order to conduct objective testing of the plastic filter fabrics, test condition consistency as well as uniformity must be maintained: (1) Consistent in that all soil types are subject to the same initial conditions; and (2) uniform in that for any one particular test, the soils are equal in size distribution and density for all permeameters. Of the two objectives, consistency was more difficult to maintain. The permeameters are all of the same size and shape. A constant head was applied to the system, equally to each of the 12 permeameters. The difficulty lay within the initial soil saturation procedure. In saturating, water flow was in an upward direction, thereby pushing the air pockets up and out of the soil sample. Every effort was made to keep the upward flow slow enough that the soil was saturated uniformly while at the same time displacing only air pockets. Even so, local piping still occurred. In testing soil types with fine particulates, it was advantageous to add a 3-in. layer of pebbles on top of the 4-in. soil column. This surcharge of pebbles significantly reduced piping effects.

The other objective that must be considered in testing is soil uniformity. The soil in each of the 12 permeameters must be of the same size distribution and density. Soil components were mixed in a small cement mixer for an 8-h period. This careful mixing of soil components which comprised a soil type, assured an approximate uniformity. Once the soil was placed within the permeameters, it was compacted to an appropriate depth that would provide a nearly maximum density. Three distinct layers of the soil column were individually compacted.

The testing procedure began with the heating and chlorination of water in the large head tank. The steam lines within the large head box and the thermostat were set so that the water temperature within the system was approximately room temperature, 68° F. Once sterilized, 25 lbs (11.3 kg) of soil was placed in each of the twelve permeameters, and compacted to a thickness of 4.16 in. (106 mm). This corresponded to a soil density of 100 lbs/cu ft (1,600 kg/m^3). The soil was then ready to be saturated. The permeameters were filled in upward direction to approximately 1 in. (25.4 mm) above the top soil surface. The saturation process usually required 8 h, depending on soil composition. Once the saturation process was completed, the transport line was connected to the permeameter inlets. Water entered the system in a downward direction and sprayed against the inside permeameter walls through the distributor heads. The inch of water above the soil depth during saturation guaranteed dissipation of the incoming water energy. This eliminated soil surface erosion and other soil disturbances. The permeameters were allowed to fill and the pressure readings to stabilize. Once the systems reached stabilized conditions, water inflow was adjusted so that a constant head was applied upon the soil depth in each permeameter. After adjustments were made, a further increment of time was

necessary for pressure and flow stabilization. When all of the permeameters indicated the required constant head, testing began.

The data collection program includes measurements of discharges and pressures. Discharge measurements were collected on a daily basis from each of the permeameter outlet taps. Once all discharge and pressure measurements were terminated, the permeameter systems were dismantled. The bottom section of the permeameter served as a sediment trap for the fine particles which passed through the filter fabric. Soil and filter samples were analyzed to determine the bacterial activity within the soil and filter fabric. The bacteria analysis was performed by a microbiologist and the tested filter fabrics were analyzed with a scanning electronic microscope to determine filter clogging or bacterial activity, or both.

Tested Soil, Filter Fabrics, and Water Head.—Four soil types were used to analyze filter fabric performance. These four soil types represent a variety

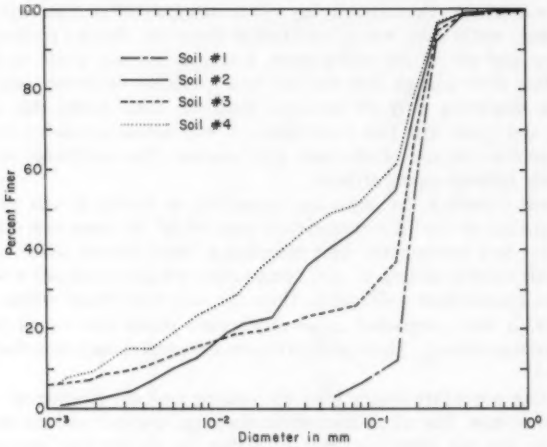

FIG. 2.—Size Distributions for Tested Soils

of fine soils which the Corps of Engineers felt to be critical to plastic filter usage. The soil size distributions are shown in Fig. 2. These soils are classified as sand (Soil Type 1), sandy loam (Soil Type 2), sandy loam (Soil Type 3), and loam (Soil Type 4), in the Triangular Soil Classification. Kaolinite was used as the clay component in testing. This type of clay was chosen for its insignificant swelling nature.

A different plastic filter was placed within each filter tube. Table 1 lists the various filter fabrics and the corresponding equivalent opening sizes used in the four soil tests. All the filter fabrics tested were nonwoven fabrics. For Soil Type 1, the hydraulic gradient (head difference between pressure Taps 1 and 6 divided by the soil thickness) imposed on the test was 2.5 ft (762 mm) of water depth upon 1 ft (305 mm) of soil. For Soil Type 2, the hydraulic

gradient simulated 3 ft of water depth upon 1 ft of soil. For Soil Types 3 and 4 the hydraulic gradient was maintained at 3 ft of water depth upon 1 ft of soil for 800 h. After 800 h the hydraulic gradient was increased 1 1/2

TABLE 1.—Tested Filter Fabrics

Soil test (1)	Filter used (2)	EOS ranges: United States standard sieve size (3)	Mean opening size, in millimeters (4)	EOS/D_{85} (5)	EOS/D_{15} (6)
1	1	80–100	0.17	0.65	1.06
	2	70–100	0.18	0.69	1.13
	3	140–170	0.10	0.38	0.63
	4	40–50	0.36	1.38	2.25
	5	80–100	0.17	0.65	1.06
	6	70–100	0.18	0.69	1.13
	7	40–50	0.36	1.38	2.25
	8	70–80	0.20	0.77	1.25
	9	40	0.42	1.62	2.63
	10	50	0.30	1.15	1.88
	11	100	0.15	0.54	0.88
	12	—	—	—	—
2	1	80–100	0.17	0.71	17.0
	2A	70–100	0.18	0.75	18.0
	3	140–170	0.10	0.42	10.0
	Z	—	—	—	—
	R	30	0.52	2.17	52.0
	7A	40–50	0.36	1.50	36.0
	8	70–80	0.20	0.83	20.0
	9	40	0.42	1.75	42.0
	10A	50	0.30	1.25	30.0
	3-01	20	0.95	3.97	95.0
	12	—	—	—	—
3	2A	70–100	0.18	0.72	24.0
	9	40	0.42	1.68	56.0
	10A	50	0.30	1.20	40.0
	3-01	20	0.95	0.80	126.7
4	2A	70–100	0.18	0.78	45.0
	9	40	0.42	1.83	105.0
	10A	50	0.30	1.30	75.0
	3-01	20	0.95	4.12	347.5

ft each day until it reached 10 ft of water depth upon 1 ft of soil. Then the hydraulic gradient was lowered to the original value.

Analysis and Consideration

Method of Analysis.—The hydraulic analysis of the filter fabrics consisted of a six-phase study. These six phases include a water filtration discharge study,

a water pressure analysis, an analysis of sediment discharges through the plastic filters, a soil size distribution analysis, an analysis of permeability changes, and a bacteria as well as a fabric fiber analysis. To avoid confusion, the following terminology is used.

1. "Filter fabric *n*" denotes the tested filter fabric identified by the Number "*n*."
2. "Filter tube *n*" denotes a permeameter device for testing the filter fabric identified by the number "*n*."

Using the measured discharges and pressure distributions, the gradient ratio and the permeability coefficients based on the average and bottom inch pressure gradients were computed.

The governing equation for soil permeability computation was given by Darcy's law

$$Q = KiA \quad \ldots\ldots\ldots\

in the soil was found to be a heavy accumulation of two bacterial types, some mold particles, and small trapped silt and debris. The bacterial types were of a sticky capsular material which could retard the water flow. The laboratory environment serves as an excellent atmosphere for bacterial activity due to the constant lighting and warm temperatures. However, city water with a chlorine residual of 1 ppm–2 ppm was used in testing. Evidently most of the residual chlorine was dissipated within the constant head tank with the remaining chlorine unable to control microbial activity below the top inch of soil. The filter cloths had some bacteria attached but under a light microscope they did not appear damaged or deteriorated. A scanning electron microscope was used to evaluate each filter fabric for possible soil or bacterial blockage within the fibers. Each filter fabric examined showed no indication of fabric clogging. Samples of the

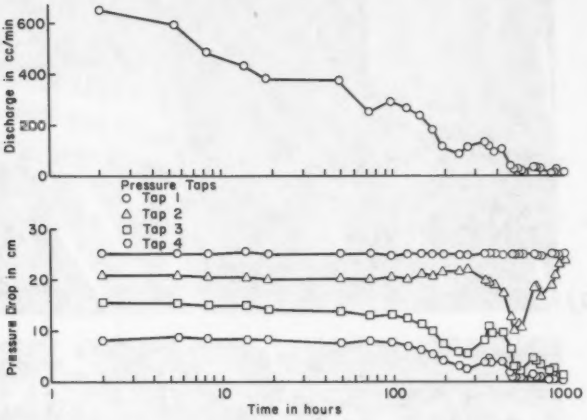

FIG. 3.—Typical Flow Rate and Pressure Distribution for Soil 1 and Filter Fabrics

microscopic photographs are shown in Fig. 4. The figure shows the control (original fabric) and one filter section both magnified 30 times. The microscopic photographs confirmed the findings of the light microscope for Soil 1 tests.

As the bacteria grew within the soil, the water velocity was greatly reduced as indicated by the flow rate decrease. This decreased the soil permeability accordingly as shown in Fig. 5. The bottom inch of soil experienced a smaller reduction in permeability than the average soil permeability as shown in Fig. 5. It is believed that the plugging of the filter system mainly occurred because of bacterial activity in the top 2 in. of soil which caused a big pressure drop across this soil layer. This is shown by the reduction in gradient ratio shown in Fig. 5. Initially the gradient ratio was larger than 1.0 due to a larger compaction in the lower portion of soil. The ratio began decreasing as the pressure drop increased within the top 2 in. of soil.

Fig. 6 is a plot of the initial and final discharges versus the equivalent opening sizes (EOS) of the filter fabrics tested. Due to data scatter no definite correlation could be initially drawn between the EOS of the filter fabrics and the water

discharges. An equation representing the weighted vertical permeability through multiple soils is

$$V = \frac{\Delta H}{\frac{d_1}{K_1} + \frac{d_2}{K_2}} \quad \ldots \ldots \ldots

in which V = the measured velocity; d_i (i = 1 or 2) = the thickness of a soil layer of a permeability; and K_i and ΔH = the water head applied to the

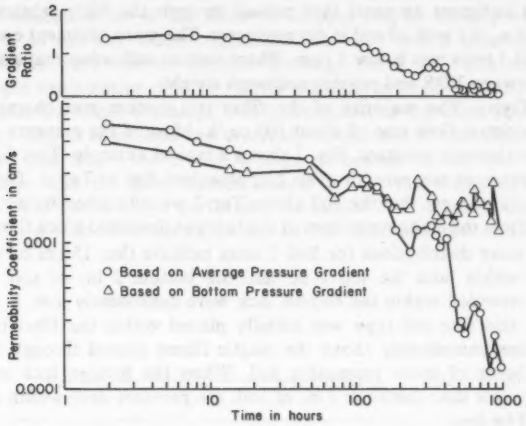

FIG. 5.—Typical Gradient Ratio and Permeability Coefficients for Soil 1 and Filter Fabrics

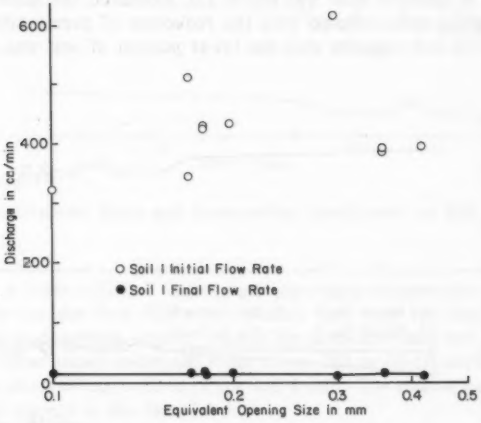

FIG. 6.—Discharge Versus Equivalent Opening Size for Soil 1 Tests

soil. The equation shows that if the thickness of an individual layer in a multiple soil system is small, the effect of this layer upon the measured velocity would be minimal. If this layer was that of the plastic filter, initially the filter fabrics would have negligible effect upon the filter system.

At the end of testing the discharge response was independent of the filter opening size. This suggested that the soil (after the growth of microorganisms) controlled the hydraulic responses of the system regardless of the type of fabric used. The sediment amounts that passed through the filter fabrics were less than 15 g, i.e., 0.1 wt% of soil in permeameter. The mean sediment concentration for the Soil 1 tests was below 3 ppm. There was no indication that a relationship existed between EOS and passing sediment weight.

Soil 2 Tests.—The majority of the filter test system was characterized by a nearly constant flow rate of about 100 cc/h. Most of the pressure differences were approximately constant. Fig. 7 shows a typical example. This figure shows that the pressures measured by Tap 2 approached that of Tap 1. The two most probable causes were that the soil above Tap 2 was disturbed by air introduced to the soil from the manometer lines or the soil was disturbed when first saturated.

The pressure distributions for Soil 2 tests indicate that 15 cm of water head were lost within both the top 2 in. and the bottom 2 in. of soil. However, pressures recorded within the bottom inch were consistently low. It is believed that when this fine soil type was initially placed within the filter tubes some fine particles immediately above the plastic filters passed through the fabrics leaving a layer of more permeable soil. When the bottom inch of soil was more permeable than the other 3 in. of soil, the pressure drop within the bottom inch would be less.

Based on the same reasoning, the permeability coefficient of the bottom-inch soil was greater than the overall permeability coefficient as shown in Fig. 8. This feature was reflected by the changes of the gradient ratio with time. The initial value of gradient ratio was below 1.0. However, the general increasing trend in gradient ratio coupled with the reduction of permeability within the bottom inch of soil suggests that the lower portion of soil was slowly being compacted.

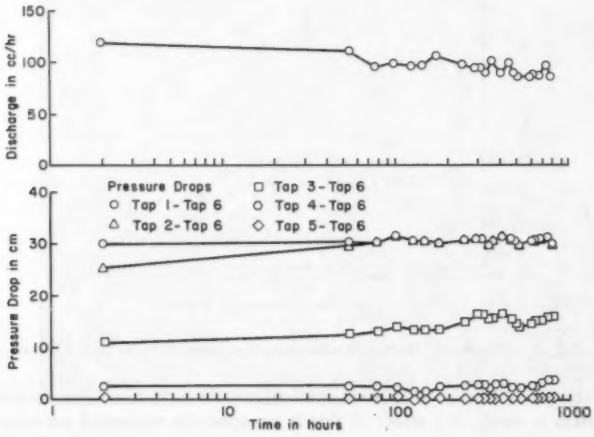

FIG. 7.—Typical Flow Rate and Pressure Distribution for Soil 2 and Filter Fabrics

Microscope examination of Soil 2 it was found that a negligible amount of bacteria existed within the soil with virtually no bacterial attachment to the filter fabrics. The bacterial activity was controlled in Soil 2 tests by boosting the chlorine concentration of the water to 10 ppm and by initially baking the soil at 400° F to kill any viable organisms. Not only was it determined that the filter fabrics were free of bacteria blockage but it was also established by microscopic examination that the fabrics were not clogged by soil particles.

The fact that the water discharge was independent of the filter fabric tested suggests that Soil 2 controlled the hydraulic responses of the filter system. It was found that Soil Type 2 with a large percentage of silt and clay was less permeable than the filter fabrics tested. Within the tested EOS ranges the plastic filters under Soil 2 performed the same. The sediment that passed the filter fabrics was not measurable due to the small amounts and very fine particulate.

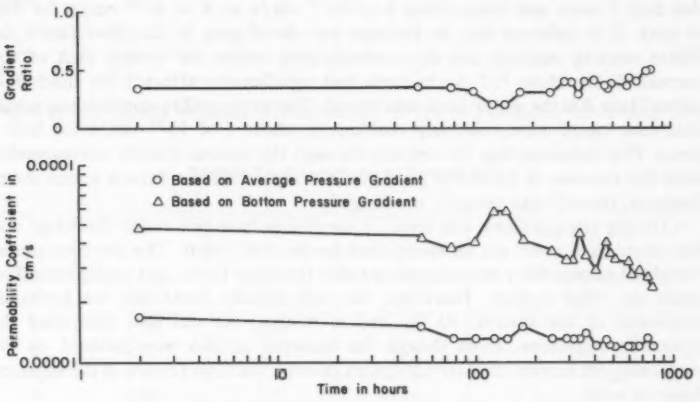

FIG. 8.—Typical Gradient Ratio and Permeability Coefficients for Soil 2 and Filter Fabrics

Soils 3 and 4 Tests.—Soils 3 and 4 tests operated at the same water head as Soil 2 tests for the first 800 h test period, and then the head increased and decreased as described earlier. During the constant head test period, the flow rates of filter tubes decreased from about 250 cc/h–50 cc/h for Soil 3 tests and from about 60 cc/h–10 cc/h for Soil 4 tests. The reason was attributed to the bacterial growth in the filter fabrics.

Upon bacterial analysis it was found that a negligible amount of bacteria existed within the soil, however a significant amount of iron bacteria existed within the filter fabrics. These bacteria originally propagated upon the metal supporter, and then grew into the filter fabric directly above. In simulating this particular bacterial growth rate in a special laboratory under ideal conditions, it was found that the bacteria developed in about 8 days. This coincided with the reduction in the filtration rates. The amount of iron bacteria present within

the filter fabrics had a significant effect on the water flow rates. The same test preparation procedure applied for Soils 3 and 4 as applied for Soil 2. However, the metal support screen used beneath the plastic filter was painted with only one coat of galvanizing compound rather than five coats. This single coating of galvanizing compound was not sufficient in preventing rust accumulation upon the metal which then supported microbial growth.

The water head was increased by raising the constant head tank after 800 h of testing. A direct correlation existed between the water head increase and the flow rate. Once the water head was lowered at 1150 h to the original height, the flow rate approached a constant value approximately equal to the discharge recorded immediately before the head increase.

The gradient ratio was initially below 1.0 for the majority of the filter tubes due to disturbance caused by backfill saturation before the test. At 80 h the gradient ratio increased above the value of 1.0 while the permeability of the bottom inch of the soil decreased from about 4×10^{-5} cm/s–2×10^{-5} cm/s for Soil 3 tests and from about 1×10^{-5} cm/s to 8×10^{-6} cm/s for Soil 4 tests. It is believed that as bacteria was developing in the filter fabric the water velocity reduced and the pressure drop within the bottom inch of soil increased. By Hour 750 the bacteria had significantly affected the discharge. After Hour 800 the water head was raised. The permeability coefficients within this time range were relatively constant at about 5×10^{-6} cm/s for Soil 3 tests. This indicates that the velocity through the system directly corresponded with the increase in hydraulic gradient. Although the filter fabrics accumulated bacteria, the soil was virtually unchanged.

Like the previous two soil tests, a correlation between water discharge and the fabric EOS could not be established for the Soil 3 tests. The aforementioned weighted permeability equation shows that the filter fabric had negligible effect upon the filter system. Therefore, the soil initially controlled the hydraulic responses of the system. At the end of testing, the soil also controlled the hydraulic responses. Even though the bacterial growth was initiated on the metal support screen, the activity spread through the filter fabrics to the adjacent layer of soil.

Analysis of Results.—A filter fabric may serve as a permeable solid constraint or a real filter. Quite often a particular soil layer within a multiple layer soil system or a homogeneous soil body, if having adequate size distributions, can work as a filter to limit the migration of fine particles within the soil. In this case, the soil layer serves as a soil filter and the filter fabric actually works as a permeable constraint to stop adjacent soil particles from washing through the fabric. Because there are few fine particles reaching the fabric, the filter fabric will not be clogged by soil. According to a filtration study, for a filter fabric to constrain large particles

$$\frac{\text{Filter Pore Size } (P_{95} \text{ or EOS})}{D_{85} \text{ (of soil)}} \leq 3 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which P_{95} = pore diameter in fabric of which 95% is finer; EOS = the equivalent opening size of a fabric; and D_{85} = the soil particle diameter of which 85% is finer. However, the Corps of Engineers suggested that

$$\frac{\text{EOS}}{D_{85}\text{ (of soil)}} < 1 \quad \ldots\ldots\ldots\

If the discharge is large enough to carry significant amount of fine particles and piles them upon the surface of filter fabric to form a cake, this cake could reduce water flow locally.

Comparing the experimental results with the Corps of Engineers' criteria described earlier, it is found that the Corps of Engineers' criteria are too conservative. The following procedure is recommended for selecting a filter fabric.

1. Determine the soil permeability coefficient from laboratory tests or from available information.
2. Determine the size distribution of soil layers to be protected.
3. Apply Eq. 7 to determine whether or not the soil layer can serve as a filter.
4. Select a range of filter fabric having permeability coefficients equal to or larger than that of soil. If the soil layer is a filter, then use Eq. 6 to select a suitable fabric. If the soil layer is *not* a filter, then use both Eqs. 6 and 8 for selecting the fabric. For the latter case, if the discharge is unusually high, then local clogging of filter fabric may occur.

It is possible that bacterial accumulations may be responsible for localized plugging of a disturbed field soil, similar to that experienced in the Soil 1 test. Many subdrainage systems contain perforated iron pipe. It has been shown in Soil Tests 3 and 4 that bacteria which metabolize iron compounds can attach to the filter fabrics and then penetrate and spread through the soil. Iron bacteria has caused drainage problems in areas with high iron content. Bacterial problems may contribute to some failures of either fabric or graded aggregate filters.

Summary and Conclusions

Plastic filter fabrics should be designed to let water pass and yet retain soil particles. Filter fabrics are relatively inexpensive and are easy to install. For these reasons they offer a possible alternative to a graded filter system. Because long-term field experience is still limited, some people are reluctant to use these new cloths. The United States Army Corps of Engineers has used filter fabrics more than any other single agency in the United States. In order to evaluate the hydraulic performance of filters, the Corps of Engineers developed their own design guidelines in 1972. Prior to this time, very little was known about the hydraulic responses of the manufactured filter fabrics. Even today, the majority of engineers who consider using plastic filters refer to the design concepts developed by the Corps of Engineers. Work at Colorado State University and other research centers is resulting in changes of design concepts.

Four different soil types were used to evaluate the filter fabric performances within this study. Each soil test consisted of measuring the water discharge and pressure distribution within a number of filter tubes. Permeameters containing a 4-in. depth of soil plus a filter fabric comprised a filter system. From the measured data, soil permeabilities and gradient ratios were computed with time.

Based on the analysis of the laboratory tests and field experience with filter fabrics, the following conclusions are drawn.

1. A natural soil is generally well graded and can serve as a filter to soil particles to limit the migration of fines. Therefore, under this condition, a filter fabric quite often works actually as a permeable solid constraint rather than a filter. In this case, clogging of a fabric by fine particles does not occur.

2. If the permeability of the soil is less than the permeability of the filter fabric and the soil is a filter which restricts the migration of fine particles, then the soil controls the hydraulic responses of the system and different filter fabrics perform the same.

3. When the soil body is not a soil filter, then significant amounts of fine particles can be carried by water through soil voids. In this case, a filter fabric works as a real filter to limit the loss of soil particles through the fabric. To avoid clogging of filter fabric it is necessary to allow the passage of fine particles. If the discharge is large enough to carry significant amounts of fine particles and pile them on the surface of filter fabric to form cake, then local clogging could occur.

4. Bacterial activity within the soil or upon the filter can control the hydraulic responses of a filter system.

5. For 1,000 h of continuous testing the filter fabrics tested had no detectable effect upon the hydraulics of the filter system.

6. The gradient ratio developed by the Corps of Engineers to indicate filter clogging should be analyzed for a longer period of time. The Corps of Engineers system could experience system instabilities that are especially prevalent in using finer soils.

7. The two basic design criteria to prevent soil movement developed by the Corps of Engineers state the D_{85} of the soil be larger than the P_{95} of the filter, determined by the EOS test, when using granular soils containing 50% or less by weight of silt and when using soils containing more than 50% of silt, the P_{95} should be smaller than the opening in the United States Standard Sieve No. 70. These criteria may apply with coarse sands with large water velocities; however, with fine sands containing more than 30% silts or nonswelling clay, or both, the soil will definitely control the system and the standard will not apply. Using a wide range of opening sizes, i.e., EOS 20 to greater than 200, with soils of large fine concentrations, very little soil will pass through the filter with little or no fine particle migration within the soil.

8. Generally speaking, the Corps of Engineers' criteria is too restrictive and conservative. A design procedure is proposed in this study which will require further verification.

Acknowledgment

This work was supported by the E. I. DuPont Chemical Company, Wilmington, Del. and Colorado State University Experiment Station, Fort Collins, Colo.

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Appendix II.—Notation

The following symbols are used in this paper:

A = cross-sectional area of soil;
C = gradient ratio;
d_i = thickness of a soil layer "i";
h_i = hydraulic gradient at lowest 1 in. of soil plus filter fabric;
h_t = total hydraulic gradient;
i = hydraulic gradient;
K = permeability coefficient of soil;
K_i = permeability of a soil layer "n";
P_{95} = pore diameter in fabric of which 95% is finer;
Q = water discharge rate;
V = measured velocity; and
ΔH = waterhead applied to the soil.

DISCUSSION

Note.—This paper is part of the Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. IR3, September, 1981. ISSN 0044-7978/81/0003-0327/$01.00.

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Optimal Seasonal Irrigation Application Analysis[a]

Errata

The following corrections should be made to the original paper:

Page 227, Eq. 18: The upper and lower limits of each integral should be interchanged.
Page 229, last sentence: Should read "Suppose one must choose between a system with $\nu = 0.20$ ($UCC = 0.84$) and $\nu = 0.10$ ($UCC = 0.92$)." instead of ". . . ($UCC = 0.75$) and = 0.10 ($UCC = 0.87$)."
Figs. 3 & 4: Should read "ν" instead of "v."

Expanded Polystyrene Spheres as Evaporation Suppressors[b]

Discussion by Keith R. Cooley[3]

The authors are to be congratulated on their efforts to reduce evaporation losses from water storage facilities. Their efforts are especially important to those in arid regions where evaporative losses of up to 3 m are common, and where this loss may be greater than the amount of stored water beneficially used. In general, the authors' presentation was clear and concise; however, the writer feels that some clarification is needed.

First, the authors indicate that their polystyrene beads did not spin and, thus, did not increase evaporation. They report that this is in contrast to results presented by Myers and Frasier (10). However, the paper by Myers and Frasier merely references another report (13), which suggests that floating beads *might* increase evaporation if the wind causes the beads to spin. Ref. 10 did not present any results or make any suggestion that their beads were spinning and, thus, increasing evaporation. The authors also state that Myers and Frasier assumed that the reflection of solar radiation by the beads was the *only* factor that significantly reduced evaporation. Instead, Myers and Frasier reported that

[a]September, 1980, by William E. Hart, Donald I. Norum, and Gideon Peri (Proc. Paper 15687).

[b]September, 1980, by Tariq A. Mahmoud, and Natiq K. Bashi (Proc. Paper 15689).

[3]Research Hydro., USDA, Sci. and Education Administration, Agricultural Research, Northwest Watershed Research Center, Patti Plaza—Suite 116, 1175 South Orchard, Boise, Idaho 83705.

reflection of solar radiation was the *major* factor in reducing evaporation. This was because the beads became wet, and the exposed wetted area was essentially the same as that of an open water surface of the same size. Therefore, the beads did not act as a true mechanical barrier, but only slightly restricted water movement to the surface of the cover, and, thus, the reflective properties became the major factor in evaporation reduction. Beads of different sizes and surface properties may perform altogether differently.

Secondly, the evaporation reduction efficiencies presented by the authors could be misleading. One should be reminded that the authors' results were obtained using very small evaporation pans under abnormal exposure conditions, and for very short time periods. Although a 6-month study was mentioned, only 3-day to 10-day results are shown. Previous studies have shown that evaporation reduction efficiencies of new material are almost always higher than efficiencies obtained for the same materials when exposed for periods of 6 months–12 months (10,14). Therefore, the efficiencies presented by the authors should be used with caution, since long-term values may be only 30%–80% of those presented.

The size of the pans used can also significantly affect results. As mentioned by the authors and demonstrated by others (14), wind can have a considerable influence on the amount of surface area covered by this type of material, and, thus, on the effectiveness of the material in reducing evaporation.

In order for evaporation reduction studies to be really meaningful, materials should be tested on tanks or ponds similar to those being used in practice to store water, and for extended periods of time. The length of the study should be determined by the expected life and cost of the cover material—it could require only one storage season or several years.

The writer hopes that the authors will continue their efforts to reduce evaporation losses of valuable water supplies, and will present the results of their long-term studies in future reports.

Appendix.—References

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UNSTEADY DRAWDOWN IN 2-D WATER TABLE AQUIFER[a]

Discussion by George Noutsopoulos,[2] M. ASCE
and Tilemachos Papathanassiadis,[3]
A. M. ASCE

The physical problem of instantaneous drawdown dealt with by the author, on the basis of assumptions described in the paper under the heading "physical model," has attracted the interest of many investigators in the past (5, 6, 7, 8, 9, and 11). All the referred investigators have approached the problem on the basis of hydraulic theory, which leads to the well-known Boussinesq differential equation. This type of treatment is quite compatible with the assumption of neglecting the seepage surface. On the contrary, the attack of the problem with the accurate potential theory has to take into account the seepage surface.

The assumptions made by the author and the restriction of small drawdown imposed by Eq. 10 are the conventional ones for dependable treatment of the problem with hydraulic theory. The resulting Boussinesq differential equation describing the physical problem in terms of the author's notation may be written as

$$\frac{\partial s'}{\partial t'} = \frac{K}{n}(h'_o - s'_o)\frac{\partial^2 s'}{\partial x'^2} + \frac{K}{n}\left(\frac{\partial s'}{\partial x'}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (41)$$

This nonlinear equation, for small slopes of the water table and small variations of s', may be linearized by neglecting the second term on the right and replacing the flow depth, $h'_o - s'$, by a mean constant value, h'_m, in the first one. Under these conditions, the linearized equation may be written as

$$\frac{\partial s'}{\partial t'} = \frac{K}{n}h'_m\frac{\partial^2 s'}{\partial x'^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (42)$$

For the instantaneous drawdown case the appropriate initial and boundary conditions are

$$s'(x',0) = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (43)$$

$$s'(0,t') = s'_o \quad \text{for} \quad t' > 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (44)$$

$$s'(x',t') = 0 \quad \text{for} \quad x' \to \infty \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (45)$$

[a]September, 1980, by David T. Higgins (Proc. Paper 15696).

[2]Prof. of Theoretical and Applied Hydr., Dir. of Applied Hydr. Lab., Civ. Engrg. Dept., National Technical Univ. of Athens, Greece.

[3]Teaching and Research Asst. in Theoretical and Applied Hydr., Civ. Engrg. Dept., National Technical Univ. of Athens, Greece.

Using the Boltzman variable $$\xi = \frac{x'}{2\sqrt{\frac{K}{n} h'_m t'}} \quad \ldots\ldots\ldots\ldots\ldots (46)$$

which is the same as the author's variable $[x^2/(4t)]^{1/2}$, in which $h'_m = h'_o$, the solution of Eq. 42 with the initial and boundary conditions (43, 44, and 45) is

$$\frac{s'}{s'_o} = \text{erfc}\,(\xi) = \text{erfc}\left(\frac{x^2}{4t}\right)^{1/2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (47)$$

This solution is the same as the authors solution for large t, Eq. 25.

The detailed study of the nature of the solution, Eq. 47, is of interest. For $x' \to \infty$, $\xi \to \infty$, and $s' = 0$. Nevertheless, for engineering purposes, $s' \simeq 0$ for $\xi = \xi_o$, in which ξ_o takes a value around 2, depending on the accuracy with which the water table measurements can be made. This means that for a certain time interval, t'_l, from the beginning, there is a certain value of the aquifer length, $x' = l'$, within which the drainage process is active. This length, l', may be defined as the active drainage length and is determined from

$$\xi_o = \frac{l'}{2\sqrt{\frac{K}{n} h'_o t'_l}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (48)$$

which is easily transformed to $$l = 2\xi_o \sqrt{t} \quad \ldots\ldots\ldots\ldots\ldots (49)$$

in which $l = l'/h'_o$.

This dependence of active drainage length with time has been tested experimentally by the writers on a Hele-Shaw model. This model consisted of two vertical parallel crystal plates 2 m long and 50 cm high, placed about 2 mm apart, with a solid horizontal bottom and open to the atmosphere at the upper end. The model was connected, upstream and downstream, to two plexiglass tanks. Commercial glycerine was used as the model liquid. The model and the tanks were initially filled with glycerine at a certain depth, h'_o. The kinematic viscosity, ν, was measured with an Engler type viscosimeter. Then a very fast drawdown of the liquid in the downstream tank, of a magnitude s'_o, was imposed with proper arrangements of the set-up. The resulting free surface in the model was observed. More specifically the time, $\sqrt{t'_l}$, from the beginning of the drawdown in the tank was recorded at each station, $x' = l'$, when an infinitesimal drawdown started at each station. More than 16 runs have been tested. For each run, the experimental results have been plotted as l' versus $\sqrt{t'_l}$, and a strongly linear dependence between l' and $\sqrt{t'_l}$ was found for all runs, as anticipated by Eq. 48. Presumably, the constant of proportionality between l' and $\sqrt{t'}$ was different for each run. Finally, the experimental data for several runs have been presented in the dimensionless form of Eq. 49, and are given in Fig. 6. In all runs $s'_o/(h'_o - s'_o) < 1$.

The experimental data confirm Eq. 49 very satisfactorily and, thus, verify the interpretation of the solution, Eq. 47, given before. For the present experi-

ments, ξ_o was found to have a value of $\xi_o = 2.175$.

The first writer and Athanasiou (8) have solved numerically the nonlinear Eq. 41, following the suggestions made by Polubarinova-Kochina (9) and compared these results with the solution of the linear one, Eq. 47. For values of the ratio $\lambda = s'_o/(h'_o - s'_o) < 1$, the differences between the results of the two solutions are small for engineering purposes. For small t', and close to the discharge point, every solution which does not take into consideration the seepage surface, always existing in the physical problem, will be in error.

Verma and Brutsaert (10) attacked the same physical problem, but for a finite horizontal extent, L, of the aquifer. Using the method of finite differences they solved the problem on the basis of accurate potential formulation, taking into account the seepage surface, and on the basis of the nonlinear Boussinesq equation neglecting the seepage surface. From the comparison of their results they concluded that, as far as $L > 4h'_o$, the results obtained by the nonlinear

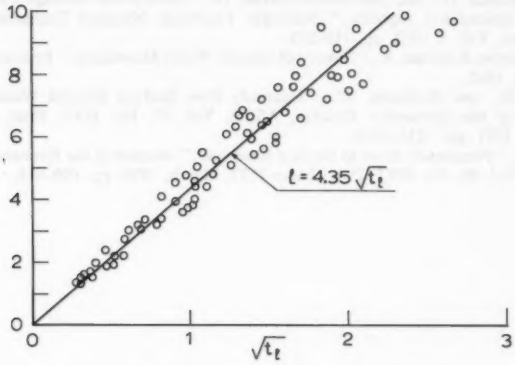

FIG. 6.—Variation of Dimensionless Active Drainage Length with Dimensionless Time

Boussinesq equation give very satisfactory results. This means that for $x' < 4h'_o$ the influence of the seepage surface is of importance.

The author presented in Fig. 5 a comparison of his experimental results with the numerical results obtained by his proposed solution expressed by Eq. 24. The writers extrapolated, also, in the whole range of t, presented in Fig. 5, the curve represented by Eq. 25, which is the same as Eq. 47. It is of interest that Eq. 25 fits the experimental data even better than the proposed more accurate Eq. 24. In reality, Eq. 25 overpredicts drawdown, which is quite logical taking into consideration that the seepage surface has been neglected in the formulation of the problem, while it is always present in nature and experiments. The contradictory result is that the proposed more accurate solution, expressed by Eq. 24, underpredicts drawdown for small t ($t < 1.0$). This underprediction should increase if the seepage surface could be taken into account. Thus, it seems logical, that while Eq. 25 gives compatible results with experimental evidence, the more elaborate treatment, presented by the author and ending in Eq. 24, does not improve the accuracy of the analytical treatment.

The author states that data from Hele-Shaw models are not appropriate. To the best knowledge of the writers, all analytical treatments of similar problems have been tested and compared with experimental data from Hele-Shaw models (10).

Appendix.—References

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